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A Methodology for Conducting Condition Surveys on Structures Constructed with Epoxy-Coated Reinforcement (ECR)

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Abstract	Epoxy-coated reinforcement has been used in bridge decks in Ontario since 1979, and in substructure components since 1981. Since the epoxy resin is not electrically conductive, one of the practical disadvantages of using ECR is the lack of effective non-destructive corrosion evaluation techniques. Condition survey methods currently used by the ministry including half cell potential measurements according to ASTM C-876 do not apply to ECR. The ministry required a methodology that could be used to effectively assess the condition of structures built with ECR and initiated a research project in 2001 to develop a protocol for conducting condition surveys on structures constructed with ECR. The project was carried out by Trow Consulting Engineers Limited, Concorr Inc., and Cement and Concrete Studies Ltd., and this publication constitutes the final report of that research project (Special Project Q 9000129).

Key Words	Corrosion, chloride ion, de-icing salts, highway bridges, reinforced concrete, condition survey, epoxy-coated reinforcement (ECR)
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A Methodology For Conducting Condition Surveys on Structures Constructed with Epoxy Coated Reinforcement (ECR)

Project Report, December 3, 2002

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Table of Contents

Executive Summary i

1. Introduction 1

1.1 Background 1

1.2 History of ECR in Ontario 2

1.3 Project Scope and Work Plan 3

2. Performance of ECR 4

3. 1988 MTO Structure Rehabilitation Manual 7

3.1 Detailed Visual Inspection 8

3.2 Detailed Condition Survey 8

3.3 Limitations of Existing Condition Survey Procedures for Epoxy-Coated Reinforcement 8

4. Non-Destructive Test Methods for Corrosion Condition Survey 10

4.1 NDT Test Methods for Detecting Delaminations under Asphalt Overlays 10

4.1.1 Ground Penetrating Radar 11

4.1.2 Impact Echo 12

4.1.3 Ultrasonic Pulse Velocity 13

4.1.4 Infrared Thermography 13

4.2 NDT Methods to Ascertain In-situ Epoxy Coating Condition 14

4.2.1 Electrochemical Impedance Spectroscopy 14

5. Condition Survey Protocol for ECR Constructed Bridge Components 16

5.1 Approach To Development of Protocol 16

5.2 Bridge Decks 16

5.3 Barrier Walls and Abutments 19

5.4 Field Evaluation 19

5.4.1 Bridge Decks 19

5.4.2 Barrier Walls 25

5.4.3 Abutments 26

5.5 Laboratory Evaluation 27

6. Service Life Predictions 31

6.1 Determining the Initiation Period, T_i 31

6.2 Determining the Propagation Period, T_p 35

6.2.1 Bridge Decks and Abutments 35
6.2.2 Barrier Walls 37
6.3 Summary 38
6.3 Summary 39

7. Condition Survey Cost Estimates **41**
7.1 Cost of Black Steel Condition Survey 41
Table 8: Bridge Deck Condition Survey, Year 2000 Costs 41
Table 9: Bridge Deck Condition Surveys, Cost Breakdown 42
7.2 Cost of Proposed Phase I Condition Survey of a Typical ECR Deck 42
Table 10: Phase I Condition Survey, Estimated Costs 42
7.3 Cost of Proposed Phase II Condition Survey of a Typical ECR Deck 43
Table 11: Phase II Condition Survey, Estimated Costs 43
7.4 Barrier Wall and Abutment Review 44
Table 12: Barrier Walls (Per Structure) 44
Table 13: Abutments (Per Structure) 45

8. Future Work **46**

9. Summary and Conclusions **48**

APPENDIX A - User's Guide to the Condition Survey Protocol, Field and Laboratory Tests

APPENDIX B - Problems Associated with Determining the Corrosion Initiation Period

APPENDIX C - Example Calculations for Service Life Predictions

REFERENCES

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Executive Summary

In February 2001 the Ministry of Transportation, Ontario (MTO) contracted with the partnership of Trow Consulting Engineers Ltd., Brampton, Ontario, CONCORR Inc. Sterling, Virginia and Cement and Concrete Studies Ltd., Toronto, Ontario, under Agreement Number 2015-A-000032, to develop a methodology for conducting condition surveys on structures constructed with epoxy-coated reinforcement (ECR).

Mr. Frank Pianca, P.Eng., Associate Research Engineer, Materials Engineering and Research Office, MTO, provided direction and guidance.

It must be understood that until recently the decks that have been surveyed are of those structures that were built prior to the introduction of increased concrete cover, improved concrete and improved waterproofing materials and the use of epoxy-coated reinforcement. This may extend the expected timing of the survey phases recommended in this report.

Key elements of the study included a review of the known corrosion protection performance of ECR including test methods to measure the condition of the coated steel reinforcement. It is postulated that condition surveys of ECR bridge decks need not be carried out prior to 20 years of service due to the improved durability of concrete structures built since the mid 1970's. However, we understand that rehabilitation work on adjacent pavements or actual replacement of asphalt on the deck itself may take place at any time from 10 – 20 years after construction. It is recommended that the Phase I survey be done at the first repaving of the deck after the bridge is at least 15 years old. The Phase I condition survey will measure the propensity for corrosion induced damage in the deck slab and will provide data required for service life prediction and the scheduling of the Phase II condition survey. The primary goal of the Phase II condition survey is to collect sufficient data to support the rehabilitation process and under normal conditions Phase II investigations will be carried out when a significant amount of structural damage has occurred. The authors recommend that the rehabilitation contract follow shortly after the successful completion of the Phase II survey to provide the best information for determining quantities and costs. The protocol for the implementation of condition surveys covers the selection and scheduling of bridge deck condition surveys, provides cost estimates for condition surveys of ECR concrete structures and includes a user's guide to the protocol and the test methods.

For Phase II condition surveys it is proposed that, in comparison to current practices, much larger areas of deck slab be exposed by the removal of the asphalt pavement and waterproofing membrane. Such procedures should lead to increased accuracy of the forecast concrete removal quantities for the rehabilitation contract documents. The proposed procedures will however require the employment of a pavement contractor since the removal, disposal and replacement of the larger quantities of pavement material will normally be beyond the capabilities of the consultant's technical crew who are completing the condition survey tasks.

The bridge deck service life prediction formulae for both the corrosion initiation period and the propagation period to a minimum amount of structural damage, represents a new approach that takes into account the influence of the waterproofing membrane on chloride penetration and the state-of-the-art knowledge with respect to the corrosion performance of epoxy coated rebars in chloride contaminated concrete. Inputs to the time to corrosion initiation period include values of depth of threshold chloride penetration and clear concrete cover of 10% above the worst case found during the survey.

The condition survey protocol, including the service life prediction, is heavily dependant on the determination of the distribution of chlorides in the concrete cover layer protecting the reinforcing steel and an increased test frequency is proposed. New test methods to be included in structure condition surveys include concrete permeability characteristics, a rating system that covers the condition of the epoxy coating on the rebars, and adhesion of the epoxy coating.

As the technology dealing with the evaluation of ECR is not very mature, the investment in this protocol will allow MTO to identify tests that do and do not correlate well with the actual condition of the ECR observed. With time we expect this procedure to be modified and the cost to reduce. This protocol has combined the use of several non-destructive tests with destructive evaluation of the condition of the ECR to provide the required correlation from actual structures. Having both provides a higher level of confidence while the technology matures.

To assist in the rapid and successful implementation of the condition survey protocol for ECR constructed concrete structures recommendations are made for future work including a training and familiarization workshop to cover the new condition survey procedures and proposals are included for the collection of statistical data from post 1978 bridge decks including distribution of chlorides, level of achieved concrete cover and condition of the coated reinforcing steel.

1. Introduction

1.1 Background

The current MTO Structure Rehabilitation Manual[1] contains procedures for the rehabilitation of concrete structures. It is divided into four parts:

Part One: Condition Surveys

Part Two: Rehabilitation Selection

Part Three: Contract Preparation

Part Four: Construction

The first part, Condition Surveys, is relevant to this project. The current MTO methodology to document the condition of bridge elements is a highly standardized procedure that has been developed over a period of almost 20 years. The procedures are practical and clear and use well proven technology and sampling/testing procedures that are familiar to much of the bridge consultant sector and construction materials testing laboratories in Ontario. Almost all structure condition surveys in Ontario are carried out by consulting engineers and construction materials testing laboratories.

The manual sets a standard, which when followed, provides accurate condition data on a structure, or parts of a structure, that supports the Ministry's bridge rehabilitation program.

Corrosion activity tests on the steel reinforcement in bridge elements are an important part of condition survey procedures and the well recognized ASTM C-876 half-cell test is used.

The introduction of epoxy-coated reinforcement (ECR) in Ontario, beginning with bridge decks in 1979, has complicated the determination of corrosion activity in the steel reinforcement. Since the epoxy resin coating is not electrically conductive the standard half-cell test method cannot be used. The Structure Rehabilitation Manual contains the statement "The corrosion potential survey should not be carried out on components with epoxy coated reinforcing steel". The problem of determining the corrosion condition of ECR and its effect on the serviceability of the bridge component is now an important issue for transportation agencies – this project addresses those needs.

1.2 History of ECR in Ontario

By the 1970's it was clear that bridge deck deterioration had become a major problem for highway authorities in all areas of North America where deicing salts were used, primarily due to corrosion of reinforcing steel that resulted in spalling of the concrete cover layer. ECR was introduced in North America in the 1970's with the objective of providing a physical barrier to chlorides at the steel/concrete interface. Research at NBS (now NIST) and FHWA indicated that coated bars performed well in salt contaminated concrete and that epoxy coated bars showed greatly reduced short-term corrosion rates, compared to black bars, in outdoor exposure plot installations. Highway authorities in North America rapidly adopted ECR and in a short time period ECR became the primary corrosion protection strategy for concrete bridge decks at jurisdictions that used de-icing salts.

Epoxy-coated reinforcement was first used in Ontario for bridge decks in 1979; essentially the longitudinal and transverse bars in the top mat only were coated. Following the appearance of significant corrosion spalling on substructure elements on a number of large, important structures in southern Ontario, ECR was specified for substructure components in 1981.

Beginning in the nineteen eighties, in a typical bridge structure, ECR is used in the following locations:

Bridge Decks:	All top mat bars. All vertical stirrups in voided, post-tensioned decks.
Abutments and Retaining Walls:	All outside face vertical and longitudinal bars on elements exposed to salt drainage or salt spray/splash.
Columns and Piers:	All vertical and transverse bars including spirals on elements exposed to salt drainage or salt spray/splash.
Column Caps and Beams:	All vertical and longitudinal bars on elements exposed to salt drainage or salt spray/splash.

The OPSS 905 Construction Specification for Steel Reinforcement for Concrete[2] covers the handling, storage and protection of ECR, the repairs to damaged epoxy coating and the post-installation inspection of coated reinforcing steel.

Until recently, Ministry policies regarding the use of ECR in bridge construction that became effective in 1979, were generally followed by the Ontario municipalities although "complete" compliance with OPSS standards was almost certainly not achieved.

The performance of epoxy-coated reinforcement is discussed in Section 2 of this report.

1.3 Project Scope and Work Plan

This project report is structured in a number of parts to address the stated key objectives and approach of the Ministry. Parts 5, 6 and Appendix A are the key technical sections.

Part 5 presents the condition survey protocol for ECR constructed bridge components that includes details of field and laboratory tests including the locations and number of tests, the interpretation of test data and the analysis of the data for service life model input. A key portion of this part of the report i.e., the determination of the corrosion condition of ECR in concrete with no visible damage, was completed following a review of published and unpublished literature and the experience of CONCORR, Inc., in U.S. research projects connected with the performance of ECR in bridge decks.

Part 6 represents the development of service life predictions to determine the remaining service life, to a minimum amount of structural damage, of bridge components constructed with ECR that includes corrosion resistance performance parameters developed from the condition survey. The predictions are based on existing state-of-the-art approaches for predicting the residual time to initiation of rebar corrosion but incorporate a new method of predicting the propagation period based on test data from the condition survey. Appendix C presents example calculations for service life predictions.

In Part 7 of the report estimates are provided for the cost of carrying out the proposed condition surveys on bridge elements constructed with ECR; such estimates are compared with the cost of condition surveys on bridge elements containing black steel using the current condition survey procedures.

Appendix A is the user's guide to the condition survey protocol. This part of the project report describes in detail field sampling, testing and evaluation procedures for all the specified tests. The laboratory storage, testing and evaluation of the field samples is described. A number of the field and laboratory tests will represent "new tests" in the MTO Laboratory Testing Manual. The level of detail in Appendix A of the report is generally suitable for inclusion in procedural or methods-of-test documents.

The final section of the report discusses the scope of future work that is designed to support the rapid and successful implementation of the condition survey protocol for bridges constructed with epoxy-coated steel.

2. Performance of ECR

Early research by two U. S. A. agencies, the National Bureau of Science, (now National Institute of Standards and Technology) and the Federal Highway Administration indicated that reinforcing bars that were cleaned and then coated with select powdered epoxies using an electrostatic spray process performed well in salt contaminated concrete [3]. Such coated rebars were determined to be resistant to corrosion, such that premature deterioration of concrete due to the pressures generated by expansive corrosion products was minimized.

Epoxy resins are thermosetting poly-addition polymers and have, for the most part, been reported to have good long-term durability in concrete and to be resistant to solvents, chemicals, and water [4]. In addition, epoxy resins generally possess the necessary mechanical properties for such service including good ductility, small shrinkage upon polymerization, and good heat resistance [5]. Further, test results have indicated that the oxygen and chloride ion permeabilities of epoxy coatings are relatively low, even in worst-case exposures [3, 4, 6]. Epoxy coatings are, on the other hand, permeable to moisture (water); and it has been known for years, although not highlighted within the highway community, that the adhesion of epoxy-based coatings to steel is reduced upon exposure to moisture [7]. Thus, the fundamental attribute by which an epoxy coating affords corrosion protection to substrate steel arises from its barrier nature, which retards ingress of chloride ions and oxygen. Also, by increasing the electrical resistance between neighboring coated steel locations, these coatings reduce the magnitude of macroscopic corrosion cell action, the latter being generally considered as responsible for premature bridge deck deterioration [8].

The Ministry of Transportation, Ontario standardized the use of epoxy-coated steel for the top mat of bridge decks and in barrier walls in 1979; and in 1981 epoxy-coated rebars were specified for some substructure components. Other provinces and agencies in Canada have also used epoxy-coated reinforcing steel. By 1975 ten U. S. state highway departments had constructed bridge decks with epoxy-coated bars and during the next decade the use of this product became widely adopted by highway agencies with freeze-thaw climates or coastline exposures (or both). Also, life cycle cost analyses utilizing epoxy-coated rebar (ECR) performance assumptions, which were considered appropriate, indicated that the increased initial cost of this material (ECR) in lieu of black steel would be recovered because of reduced latter life maintenance expense [9]. That this technology has been widely accepted is exemplified by the fact that the Ministry of Transportation and many provinces and most U.S. state highway agencies now have a considerable inventory of bridges with ECR [10].

The initial developmental and field investigations of ECR performance were generally positive with regard to the protection afforded to embedded steel and the utility of this methodology for corrosion control [5, 8, 9, 11-25]. Subsequent laboratory and outdoor exposure studies were also generally positive [26, 27]. Beginning with the disclosure that the splash zone region of ECR substructure elements in Florida Keys bridges exhibited corrosion induced concrete cracking and spalling after only six to nine years exposure [10, 28-31], many questions

pertaining to the corrosion resistance of ECR in concrete were raised, however, they have been readdressed by several comprehensive studies [32-34]. It was the general conclusion from these studies that, although concrete damage from ECR corrosion was not presently widespread, this corrosion control alternative was not likely to provide the relatively maintenance free service life that was forecast earlier. In addition, extensive repairs and rehabilitations would ultimately be required during the anticipated service life of many structures. In support of this, the ECR corrosion induced distress that has been observed in the field [10, 28-33,35-36] has been reproduced in the laboratory and test yard and mechanisms for its occurrence have been identified [32-34]. The overall North American experience with the corrosion performance of ECR to the mid 1990's has been summarized by Manning [37].

Two mechanisms for the failure of epoxy-coated rebars have been proposed. The first suggests failure by adhesion reduction. Adhesion reduction or disbondment can result from one or more of the following mechanisms:

1) wet adhesion loss
2) anodic undermining
3) cathodic disbondment

A regional study evaluated the condition of 473 epoxy-coated rebars extracted from 240 cores which were collected from 80 bridge decks located in the states of New York and Pennsylvania and probability distribution analysis of coating adhesion results indicated that more than 50% of the epoxy coating experiences some degree of adhesion reduction within 6 to 10 years of placement in concrete [38]. However, this study did not find coating adhesion to be a good indicator of corrosion condition of the epoxy-coated rebars. Similar findings were also reported by a study that evaluated a total of 109 cores from the province of Alberta and the cities of Calgary and Edmonton [39]. Other researchers have proposed that disbondment itself is failure of the coating and a disbonded coating does not provide any protection in the presence of chloride ions [40]. Further, the report by the University of Waterloo prepared for the Ministry of Transportation suggests that active corrosion will eventually occur in the disbonded regions of the epoxy coating even in the absence of chloride ions [41]. Weyers et al. [40] proposed that under such circumstances, the lack of oxygen and the neutral or acidic solutions within the epoxy will allow the corrosion to take place without the production of voluminous corrosion products and the associated cracking and spalling of the cover.

The second failure mechanism proposes failure from coating defects. A recent study completed by Hartt et al. [42] concludes that a) coating defect density was the predominant parameter that affects ECR performance, b) the number of coating defects increase with exposure time and the rate of this increase being constant and proportional to the initial defect density for each exposure type, and c) coating disbondment occurs at and proceeds outward from coating defects. These findings are supported by a study of the Florida Keys bridges performed by Saqüés et al[43]. In this study, Saqüés and coworkers developed service life models to fit the findings on the Florida Keys bridges and concluded that the quality of

the coating controls the propagation phase of the corrosion process on epoxy-coated rebars. A study funded by the Ministry of Transportation, Ontario to evaluate the failure of a section of the Perley Bridge deck also suggests that the poor quality of the coating (electrically porous) was responsible for the early failure of a section of Span 17. This means that the better the quality of the coating the longer it takes to develop corrosion induced damage.

Based on the literature survey, the authors of this report believe that the second failure mechanism seems to be supported by a larger body of field and laboratory data both from Canada and the United States.

3. 1988 MTO Structure Rehabilitation Manual

The Bridge Deck Rehabilitation Manual was first published in 1983 and consisted of two parts: Part One, Condition Survey and Part Two, Contract Preparation. Since the manual was written, a number of changes in deck rehabilitation procedures have been implemented and procedures for rehabilitating other concrete components have been developed.

The Structure Rehabilitation Manual, 1988, superseded the Bridge Deck Rehabilitation Manual and included procedures for the condition survey and rehabilitation of all above-grade concrete components of highway bridges. It is divided into four parts:

Part One:	Condition Surveys
Part Two:	Rehabilitation Selection
Part Three:	Contract Preparation
Part Four:	Construction

Part One: Condition Surveys, describes how condition surveys are to be carried out. Appendices 1.A to 1.D include consultant agreements, standard forms and standard legends. Consultants normally carry out condition surveys, however, the Ministry makes recommendations for the rehabilitation and the decision on the method of rehabilitation in-house. The contract documents may be prepared in-house or by a consultant. Other authorities, especially municipalities, frequently engage consultants to carry out the condition survey, make recommendations for repair, prepare the contract documents and supervise the construction. Occasionally, one consultant may carry out all the activities but frequently two or more consultants will be involved.

The information contained in the manual is based largely on the Ministry's Bridge Deck Rehabilitation Manual, which it replaced, and on the Ministry's Research and Development reports published between 1975 and 1988 as well as the Ministry's experience in preparing and administering structure rehabilitation contracts during that period. The current manual has been updated many times with various sections being dated Jan., 1990; May, 1990; Dec., 1991; Feb., 1993 and July, 1995.

Condition surveys on bridge components have involved carrying out a detailed visual inspection of the structure and detailed condition surveys of the various structure components. The type of survey required is determined by the proposed rehabilitation and this in turn controls the extent and detail of data to be collected. The purpose of the survey is to determine and document the deterioration in the structure so as to establish the type of rehabilitation and then to prepare contract documents. The visual and condition surveys are described in the Structure Rehabilitation Manual as follows:

3.1 Detailed Visual Inspection

"A detailed visual inspection of all components according to the procedures given in the Ontario Structure Inspection Manual, OSIM, may be required, to determine if repairs of these components should be included in the rehabilitation contract. However, caution should be exercised when assessing the overall condition of the component, as the visual observations may not reveal hidden defects or deterioration in the concrete such as delaminations, rebar corrosion and low concrete cover to reinforcing steel."

3.2 Detailed Condition Survey

"For black steel reinforced components a detailed condition survey is carried out only after a concrete component has been identified for rehabilitation. The data collected is then used to establish the rehabilitation method and to prepare contract documents. The procedure for carrying out a detailed condition survey involves the observation and recording of surface defects and may also involve a delamination survey, a cover meter survey, a corrosion potential survey, coring of concrete components, asphalt sawn samples and physical testing of the concrete cores.

Candidates for a detailed condition survey will include deck slabs included in paving contracts that have not been rehabilitated or constructed in the last 15 years. Other candidates are decks that are known to require rehabilitation for other reasons identified during other inspections.

The detailed condition survey is normally carried out no more than two years prior to the proposed rehabilitation. Where a project is deferred, so that the detailed condition survey is more than four years old at the time of construction, it may be necessary to update the original survey.

The existing condition survey protocol has an escalation clause for additional testing where the deck exhibits high corrosion potential or a large quantity of delaminated concrete. There is also a difference in the amount of coring and number of sawn samples between a First Time Survey and an Update Survey."

3.3 Limitations of Existing Condition Survey Procedures for Epoxy-Coated Reinforcement

The detailed condition survey procedures for ECR bridge components will require some major changes. These changes are due to some fundamental differences in the reinforcing steel and how the amount of corrosion and even potential corrosion can be determined. In properly constructed ECR decks the use of the standard corrosion potential equipment is not possible due to the electrical insulation of the reinforcing members. This poses a serious problem in determining the corrosion state of the deck and requires significant changes to the condition survey protocol.

As stated in the Structure Rehabilitation Manual, "A detailed condition survey is carried out only after a concrete component has been identified for rehabilitation." With the use of waterproofing systems and the replacement of these systems every 15-20 years on average, the life of deck components has been extended. The use of ECR in decks could extend the life of the component if the epoxy coating is of good quality and the surrounding concrete is also of good quality. There is a need to determine the condition of these decks and to predict the future rehabilitation needs during the life of the deck.

The use of a phased condition survey which is described herein will provide information on the progression of chlorides into the deck slab and the condition of the epoxy coating that is meant to protect the steel once chlorides reach the steel/concrete interface. This evaluation will be carried out prior to membrane replacement projects every 15 – 20 years. The Phase 1 survey will provide detailed information on concrete cover, concrete permeability, epoxy coating properties and chloride concentrations at various depths. All of the information gathered will be used to estimate the time to critical chloride levels at the steel and the anticipated time to corrosion related damage.

The timing of the surveys will be affected by work on adjacent structures. The authors understand that if the adjacent pavement is being rehabilitated before the normal timing of the deck rehabilitation then that would trigger work on, or at least a review of the bridge components.

4. Non-Destructive Test Methods for Corrosion Condition Survey

The primary goal of any condition survey is to obtain the maximum number of data points, in the least amount of time, while minimizing damage to the reinforced concrete element. This can be best accomplished and in some cases can only be accomplished by the use of appropriate non-destructive test (NDT) techniques or methods. Two areas, in which the NDT techniques could be very effective on ECR reinforced concrete elements especially bridge decks, are a) detection of delaminations under asphalt overlays and b) ascertaining the in-situ condition of the epoxy coated rebars.

As sounding and visual surveys cannot be usefully performed on bridge decks overlaid with asphalt, either a NDT test method has to be used or the asphalt overlay and the waterproofing membrane have to be removed and replaced. The cost and logistics associated with the removal and replacement of the asphalt overlay and the waterproofing membrane normally limits the survey to select sections of the deck, thereby, limiting the accuracy of the estimate of the total quantity of damage. A certain degree of accuracy in the estimate of the total quantity of damage is essential in maintaining budgetary control over the repair and rehabilitation project. If a NDT process can accurately measure the concrete damage (i.e. delaminations) under the asphalt overlay and the waterproofing membrane over the entire deck surface it would significantly enhance the ability to accurately estimate the total quantity of repair for the construction project and improve the accuracy of the derived projections. Numerous such techniques have been proposed.

The condition of epoxy rebar is generally ascertained by extracting ECR in cores. This results in damage to both the concrete and the reinforcing steel and also compromises the integrity of any waterproofing membrane, if present. The damage and the cost associated with the extracting and testing, limits the quantity of data collected, thereby, limiting the accuracy of projections developed from such data. A NDT test method would significantly improve the quantity and quality of ECR condition data.

A literature survey was performed to identify such methods applicable to ECR reinforced concrete elements, especially bridge decks.

4.1 NDT Test Methods for Detecting Delaminations under Asphalt Overlays

Each of the techniques discussed within this section are considered to hold the most promise for assessing the condition of concrete bridge decks that are covered by a waterproofing membrane and asphalt overlay. Although these techniques show promise and may have a history of use on bare decks (no overlay), the addition of the membrane and asphalt overlay significantly reduce the effectiveness of each technique. For each technique, the addition of the membrane and overlay either reduces the ability to distinguish anomalies within concrete

or completely absorbs the energy (radar, stress wave by impact, ultrasonic, or thermal) such that the resultant reduction in accuracy is considered prohibitive.

4.1.1 Ground Penetrating Radar

Ground Penetrating Radar (GPR) operates by directing electromagnetic waves toward an object of interest. As the electromagnetic waves pass through the object, energy is reflected at boundaries between two materials possessing differing values of the dielectric constant (or real part of the complex conductivity). While some energy is reflected at each boundary, the remainder continues propagating into the object. Typical dielectric constant values seen in civil engineering applications include: air = 1, concrete = 8-12 (depending on material properties and proportions of constituents), and water = 81. More energy is reflected at an interface between two materials where the difference in the dielectric constant is greatest. Thus, it is easier to detect (more energy is reflected) a defect such as a water-filled delamination (water/concrete interface) rather than a defect such as an air-filled delamination (an air/concrete interface).

The typical frequency of operation for GPR varies from 80MHz to 2GHz with the higher frequency antenna being more suited to bridge deck studies. An antenna of higher frequency will result in greater resolution (ability to differentiate between objects) but lesser depth of penetration. Several research studies have shown GPR to be an effective tool in monitoring the subsurface condition of bridge decks. Two research studies [45, 46] utilized GPR to locate defects and reinforcement bars. A third study [47] reported using GPR to successfully determine the cover depth of concrete bridge decks.

Ground Penetrating Radar (GPR) has been evaluated by several researchers for various applications to the analysis of civil infrastructure. However, as with many technologies, its usefulness may have been oversold before the limitations of this particular technology were fully understood. While the above researchers have reported success in their application, GPR is not yet a tool that can be guaranteed to work for all applications. For instance, surface moisture can significantly hamper the ability of the electromagnetic waves to penetrate the surface of a structure. Significant amounts of reinforcement steel in the bridge deck can reflect much of the initial signal strength thereby obscuring reflections from below. For concrete bridge decks containing an asphalt overlay, air gaps or moisture between the concrete and asphalt can also reflect much of the initial signal strength, thus reducing the energy that can penetrate into the deck below.

Commercially available GPR packages are automated to various degrees. Such packages allow the survey to be performed from a moving vehicle at speeds of 10 to 30 km/h and do not require closure of traffic lanes during the survey. With an increase in speed comes a decrease in the resolution capacity of the system.

Interpretation of GPR data requires a certain level of knowledge and training which is not readily available. Several vendors have attempted to automate the process of data

interpretation. Our experience suggests that these first attempts at automation have not yet reached a sufficient level of maturity to provide reasonably accurate interpretation of data. CONCORR, Inc. evaluated one of the commercially available GPR equipment packages for 2 years on numerous bare and asphalt overlaid bride decks and observed that it did not provide sufficiently accurate results on all structures. The accuracy of the delamination survey varied significantly from one structure to another. Therefore, presently available GPR packages are not recommended for use in estimating the quantity of damage on bridge decks with or without asphalt overlays.

4.1.2 Impact Echo

Impact Echo is a process in which a mechanical impact produces a stress wave in a material. This stress wave travels through the material until a discontinuity is encountered. At the discontinuity, a portion of the stress waves is reflected. For any isotropic and elastic media, there are three modes of stress wave propagation: dilatational, distortional, and Rayleigh waves. Dilatational waves (P-waves or compression waves) produce particle motion that is parallel to the direction of stress wave travel. Distortional (S-waves or shear waves) produce particle motion that is perpendicular to the direction of stress wave travel. Rayleigh waves (R-waves) travel near the surface of the material in a retrograde elliptical motion.

Typically, a mechanical impact is produced by striking a small (diameter less than 5mm) metal sphere on the surface of the material to be tested. This impact produces a stress wave, which propagates in all directions. As this stress wave propagates into a material, reflections occur at interfaces between two materials of differing acoustic impedance. A transducer, located at the surface near where the impact initiated, can receive these reflected stress waves. By measuring the time difference between the initial impact and the receipt of reflected stress waves from a boundary, the depth to that boundary can be estimated. It is important to have a basic understanding of the velocity of the stress wave traveling through the material. A typical P-wave velocity of 3000-4500 m/s is assumed for concrete (depending on the proportions and specific gravities of the constituents). This value can also be measured by performing impact echo where the depth to a boundary or thickness is already known.

The resolution and depth of penetration are controlled by the impact duration or contact time between the metal impactor and the surface of the material. The impact duration is proportional to the diameter of the impactor; shorter duration impacts are produced with impactors of smaller diameter. In addition, the frequency of the stress wave is inversely proportional to the impact duration; an impact of shorter duration produces a stress wave of higher frequency. Thus, an impactor of small diameter will produce a higher frequency stress wave. Much as the frequency of the antenna affects the depth of penetration and resolution of GPR, a higher frequency stress wave will provide greater resolution but lesser depth of penetration.

Impact echo can be a useful technique given the proper conditions. The surface texture of the material to be tested is important to consider. Although impact echo can be used to perform analysis on hot-mix asphalt (HMA) pavements, due to the typically rough surface texture, it's usefulness in this application may be limited. Impact echo, however, has been employed as a useful technique to monitor the condition of various concrete structures[48,49].

The accuracy and reliability of impact echo in detecting delaminations on asphalt-overlaid decks has not been sufficiently documented in the literature. There is concern that the majority of the impact energy will be absorbed by the asphalt overlay and reflected from the asphalt waterproofing membrane interface. Thus, impact echo is not recommended for conducting delamination surveys of asphalt overlaid bridge decks.

4.1.3 Ultrasonic Pulse Velocity

Ultrasonic pulse velocity works in much the same manner as Impact Echo. The primary difference is in the stress wave that is applied. While Impact Echo employs a stress wave (resulting from a mechanical impact), Ultrasonic Pulse Velocity uses a high frequency (greater than 20kHz) sound wave emitted from a piezoelectric transducer. The pulse velocity method employs two transducers, one to send and one to receive the sound wave. Using the two transducers, the travel time is calculated. If the thickness of the material is known, the travel time can be converted into velocity and compared with typical values (approximately 3,700 m/s for concrete). By comparing the calculated velocity with typical values, a determination can be made about the condition of the material under study.

Several factors may affect the accuracy of the Ultrasonic Pulse Velocity method [50]. Proper contact between the transducers and the surface of the material being tested is critical; thus, a couplant is used to ensure that there is not an air gap between the surface of the material and the transducer. The temperature and the moisture content of the concrete may also play a small role in affecting the travel time of the ultrasonic pulse. In addition, the minimum spacing between the transducers (thickness of material being tested) should be greater than approximately 100 to 150mm. This value will depend upon the velocity of the ultrasonic pulse in the material and the frequency of vibration of the piezoelectric material in the transducer.

As with Impact Echo, there may be some difficulties in using Ultrasonic Pulse Velocity on an HMA surface. Again, given the rough surface texture typical of HMA pavements, it may be difficult to establish a proper contact with the surface in order to perform the ultrasonic testing.

4.1.4 Infrared Thermography

Infrared Thermography is based on the principle that defects within a material will alter the way heat flow is dissipated at the surface of that material [51, 52]. These changes in surface temperature can be measured to locate and possibly to determine the quantity of subsurface

defects. Heat flow will occur when the temperature of the material differs from the temperature of its surroundings.

A common practice when using Infrared Thermography on large structures is to perform the testing early in the morning while the sun is heating a structure or just after dark when the energy from the sun is being released into the environment. Since concrete structures typically involve large areas, natural sources of heat (such as the sun) can be useful in performing Infrared Thermography. However, it has been stated [53] that this form of passive heating tends to reduce the ability to resolve the dimensions of specific defects since radiation from the sun is not very strong and thus it takes a significant amount of time to provide enough heat to flow through a large structure. ulain *et al.* [53] present research where reinforcement in concrete slabs is heated resistively. There are several factors, which can influence the accuracy of Infrared Thermography outside the laboratory setting. These factors include: surface texture, wind speed, and surface moisture. Each of these factors can influence the way that heat is adsorbed or dissipated.

CONCORR, Inc. has evaluated Infrared Thermography on several bare decks and developed a complete package of Infrared equipment and software to collect and process the data to identify the size and location of each delamination. The use of this equipment met with limited success. Due to a loss of resolution when employing the transfer of heat through multiple materials, Infrared Thermography is not applicable to asphalt overlaid bridge decks.

4.2 NDT Methods to Ascertain In-situ Epoxy Coating Condition

4.2.1 Electrochemical Impedance Spectroscopy

Electrochemical Impedance Spectroscopy (EIS) measures the complex impedance (containing both real and imaginary portions) of the interface between concrete and internal reinforcement over a wide frequency range (typically less than 1MHz). As the formation of corrosion products on the surface of a reinforcement bar occurs from dissolution and precipitation of ions in the pore water solution, the corroding surfaces are covered by porous films [54]. These porous films may contain insulating, partially conductive, or even high conductive materials; all of which affect the impedance characteristics across the corroding surface. By measuring the impedance of this circuit (containing the reinforcement, coating, and the surrounding concrete) an estimate of the condition of the coating surrounding the reinforcement can be given [54]. An increase in the ability of a current to conduct across the reinforcement coating indicates a reduction in the protective nature of the coating. Although, it is possible to have a slight increase in the current flow across the reinforcement coating without a significant loss in protection, this point signals the start of the process leading to eventual breakdown of the coating and loss of protection for the reinforcement [55].

Although EIS shows promise in laboratory related studies [56], it may prove to be too cumbersome for field use. Electrochemical Impedance Spectroscopy incorporates expensive equipment, necessitates direct contact with the reinforcement bar, requires a highly trained

operator with sufficient experience to interpret the results, and can consume large quantities of time due to location of and connection to bars. These drawbacks have thus kept EIS primarily in the laboratory. However, recently published research has been presented involving a surface measurement EIS method that may hold some promise for assessing structures in the field [57].

A study of the EIS technique is presented in a report published by NCHRP [55]. The method described in this report requires the operator to perform EIS measurement over a wide frequency range and use the impedance measurement at 0.1 Hz as an indicator of the coating condition. CONCORR, Inc. has applied this technique on several field structures and on 178 cores with two intersecting epoxy coated reinforcement bars, which were removed from 18 bridges. The condition of each of the bars was also visually rated using a condition rating scheme. The correlation constant (r^2) between EIS and condition rating, number of holidays, quantity of coating damage and coating thickness was less than 0.1. Thus EIS is not recommended for evaluating the in-situ condition of ECR.

5. Condition Survey Protocol for ECR Constructed Bridge Components

5.1 Approach To Development of Protocol

Most bridge maintenance programs only require a condition survey prior to a major repair or rehabilitation of the structure. During the development of the methodology for evaluation of concrete elements reinforced with ECR it was realized that a somewhat different approach was required for bridge decks in Ontario. Therefore, the protocol proposed below is somewhat in the form of a bridge maintenance program but requires two different types of condition surveys to be performed on bridge decks every 15-20 years. The approach to abutments, barrier walls and other "exposed" bridge components remains the same, i.e. perform a condition survey prior to repair.

In general, the primary goal of any condition survey is to collect sufficient information to accurately estimate repair quantities and to identify the optimal repair and rehabilitation option. Repair quantities are estimated by assessing the half-cell potentials, quantity of cracked, delaminated, and spalled areas on the structure. To select an optimal repair and rehabilitation method, information on future damage and remaining service life is required. The quantity of damage at some point in the future can be estimated by modeling the process of corrosion induced damage generation.

The process of corrosion induced damage generation is generally modeled in two phases: time to corrosion initiation and time for propagation of damage. The time to corrosion initiation is the time required for the threshold quantity of chloride ions to accumulate at the steel/concrete interface to initiate corrosion. The time for propagation of damage is the time required for damage to precipitate after the chloride ions have initiated corrosion. The time to corrosion initiation is dependent on the diffusion characteristics of the concrete element and the condition and effectiveness of the waterproofing membrane system (if present). The time to propagation is primarily dependent on the quality of the ECR. To estimate future damage, information on the distribution of chloride ions within the structure, the distribution of clear concrete cover, the quality of the epoxy coating, the condition of the reinforcement, and the permeability of the concrete is required. Inputs to the time to corrosion initiation include values of depth of threshold chloride penetration and clear concrete cover of 10% above the worst case found during the survey.

5.2 Bridge Decks

The quantity of damage can be easily assessed on elements, which have exposed concrete surfaces, such as barrier walls and abutments, using visual and sounding surveys. However, bridge decks having a waterproofing membrane and an asphalt riding surface do not easily lend themselves to such assessments. A waterproofing membrane and an asphalt riding

surface installed during construction protects all ECR decks in Ontario. The presence of the asphalt riding surface and waterproofing membrane prohibit both a visual and a nondestructive assessment of the condition of the deck concrete and thus present a challenge in determining the condition and need (if any) for repair.

Prior to conducting any detailed condition evaluation (requiring substantial destructive testing in the form of a core or ECR extraction), the susceptibility of the deck to corrosion induced damage needs to be established. Unfortunately, even the evaluation of susceptibility to corrosion requires some destructive testing, albeit, significantly less in magnitude than a detailed survey. For decks with insufficient chloride ion concentration at the steel/concrete interface to cause corrosion, the results of the susceptibility evaluation help avoid unnecessary testing and evaluation and minimize the amount of damage to the waterproofing membrane. Although, damage to the waterproofing membrane in sampling areas can be repaired, the original effectiveness is often not restored and thus testing that requires removal of the membrane should be used judiciously.

An optimal approach to bridge deck maintenance would be to subdivide the condition survey into two phases; Phase I to identify the susceptibility of the deck to corrosion induced damage and Phase II to obtain sufficient information for the development of repair plans. If the results of Phase I indicate that the deck is susceptible to corrosion damage in the short term, it can be followed by Phase II at an appropriate time. Considering that bridge decks with ECR in Ontario are constructed with an effective waterproofing membrane, increased cover, and generally good quality concrete, it is recommended that the first Phase I testing be performed after the bridge deck has been in service for 20 or more years and then repeated every 15 years. If a repaving is scheduled to occur on the bridge deck prior to completion of 20 years of service, Phase I evaluation should be performed during or just prior to repaving. Phase II should follow Phase I only if the results of Phase I suggest that the deck is expected to suffer damage in the near future.

The primary goal of the Phase I survey is to determine when corrosion has either initiated or will initiate in the future and when the resulting damage can be expected. This is accomplished by the use of diffusion modeling to estimate the time required for the concentration of chloride ions at the steel/concrete interface to exceed the threshold of $0.71 kg/m^3$ ($1.2 lb/yd^3$) or approximately 0.025 % (250ppm). If diffusion modeling indicates that sufficient chloride ions have accumulated at the steel/concrete interface and corrosion may have initiated or is expected to initiate in the next 15 years, then the quality of the ECR shall be used to estimate the propagation time. The time to propagation has been linked to the condition of the epoxy coating in two recent studies, one by the National Cooperative Highway Research Program (NCHRP) [42] and the other by the Florida Department of Transportation [43]. To minimize damage to the waterproofing system, Phase I testing is limited to collection of 100mm diameter core samples (with and without ECR extracted in the core). A laboratory evaluation of the cores is proposed to provide information on clear concrete cover, distribution of chloride ions, the condition and quality of ECR, and the permeability of the concrete. The results of the laboratory evaluation are used to estimate

time to corrosion initiation and propagation using the service life model developed specifically for ECR decks in Ontario. The decision matrix provided below can then used to determine future action. The decision matrix is based on the assumption that repaving projects, possibly including replacement of the waterproofing membrane, are carried out approximately every 15 years.

Results of Phase I Condition Survey and Service Life Prediction	* Actions to be Taken
Time to Corrosion Initiation > 15 years	Repeat Phase I survey at next repaving cycle (~15 years)
Time to Corrosion Initiation < 15 years	Perform Phase II evaluation at time of anticipated damage or 2 years prior to planned rehabilitation

The primary goal of Phase II is to collect sufficient data to select the optimal repair procedure and protect the structure from future deterioration. The most important data to be obtained in this phase is the quantity of damage suffered by the deck. For proper selection of the protection system, it is also essential to know the probability and the extent of expected future damage. It is suggested that a Phase II survey be carried out just before the deck is planned for rehabilitation to provide the most current information for the rehabilitation contract. Therefore, Phase II of the protocol is carefully designed to estimate the quantity of present damage, present condition, quality of the epoxy-coated rebar and the propensity for future corrosion and subsequent damage. In this phase, substantial sections of the asphalt overlay and underlying waterproofing system are removed to conduct a condition survey. Within each exposed area a damage survey, including delaminations, electrical continuity testing, half-cell potential survey, and core sampling is proposed. ECR shall be extracted in all cores except those designated for permeability testing. Laboratory evaluation of the cores shall include chloride ion distribution, clear concrete cover, condition and quality of ECR, and permeability of concrete.

Electrical continuity testing provides information on the quality of the construction; i.e., the level of damage inflicted on ECR before and during construction. An ideal coating (undamaged during construction) would result in electrically discontinuous ECR. However, damaged ECR (due to improper construction) provide some degree of electrical continuity. The DC electrical continuity technique can detect the presence of micro and macro cell corrosion currents although their magnitude cannot be measured in a bridge deck. Half-cell

* It is understood that bridge deck pavement systems i.e., waterproofing membranes, protection board and asphalt layers are generally replaced at 15 years. There are clearly economic benefits to the Ministry if the Phase I condition surveys are carried out at the time the deck pavement system is replaced. When this is not possible, or advisable, and indeed this may be the case for most bridge decks, it will be valuable for the purposes of future rehabilitation planning if some basic tests such as delamination sounding and chloride testing of core samples are carried out during the time the pavement system is replaced.

potential measurements can be made on ECR if a positive connection is made to the bar subjected to the measurement. Although the criterion advocated by ASTM C-876 (procedure for bare uncoated bars) cannot be used to analyze half-cell potential data for ECR, the presence of potentials more negative than -350mV versus copper-copper sulfate indicate a break in the coating and/or active corrosion on the ECR. Therefore, half-cell potential measurements can be used to detect deteriorated areas on an ECR bar. Thus, to limit the number of cores required, half-cell potential measurements shall be used to obtain additional information on the condition of the coating. After excavating a section of a bar, such that an electrical contact can be established, half-cell measurements shall be performed every 150mm along the length of each bar within the area of exposed concrete (sawn areas).

The damage quantities such as surface delaminations obtained from the sawn areas can be projected to the entire deck surface by simple extrapolation. The results of the other evaluations can be used to select the optimal repair and protection system. As the service life model developed under this project has not been validated on actual structures, it is considered more appropriate for use in Phase I than in Phase II. Therefore, in addition to the service life model, the results of all tests should be independently analyzed to ascertain the most optimal repair and protection system for the bridge deck. The results of the Phase II analysis can be used to verify or revise the service life equations for future analyses.

5.3 Barrier Walls and Abutments

A protocol was developed to evaluate the condition of barrier walls, abutments and other "exposed" bridge components. It is based on the premise that a condition survey will be performed on these elements when sufficient corrosion induced damage is visible and repair and rehabilitation is planned. The goals, test methods, and the level of testing are very similar to that of Phase II for bridge decks. Sufficient testing is proposed to allow the use of the service life equations and conventional analysis of data to select the repair and protection system.

5.4 Field Evaluation

5.4.1 Bridge Decks

It is recommended that Phase I testing be performed on bridge decks 20 or more years old and then repeated every 15 years. Phase II should follow Phase I only if the results of Phase I suggest that the deck is expected to suffer damage in the near future. Phase II testing should be performed 2 years prior to deck repair.

Phase I

To minimize damage to the waterproofing system, Phase I testing is limited to collection of 100mm diameter core samples and performance of laboratory evaluation. The number of

cores, sampling locations, and location distribution shall be in accordance with Tables 1 and 2, respectively. Two types of cores are to be collected: cores containing ECR (ECR cores) and cores without ECR (non-ECR cores), mainly for chloride determination. The rapid chloride permeability (RCPT) cores are additional non-ECR cores collected specifically for permeability testing.

Table 1: Frequency of Core Sampling

Deck Area (m^2)	# of Cores with ECR	# of Cores without ECR	Rapid Chloride Permeability Cores
<1,500	5	5	3
1,500 to 3,000	1 per 200m^2	1 per 200m^2	3
>3,000	15	15	3

Table 2: Core Locations and Distributions

Sampling Locations	Core Locations	Percent of Cores with ECR	Percent of Cores without ECR
Curb	From a 0.5m wide strip of pavement along the curb	20%	20%
Crown of Deck	From a 0.5m wide strip of pavement centered along the crown of the deck	10%	10%
Traffic Lanes	Within the traffic lanes	20%	20%

To extract two intersecting ECR in an ECR core, the ECR have to be precisely located. If the ECR intersections can be located directly from the asphalt surface, this is the preferred method. However, this may be difficult to accomplish in the presence of a thick layer of asphalt overlay and high concrete cover with commercially available rebar locators. To overcome this problem, a 500mm x 500mm area shall be marked on the surface of the bridge deck for removal. The asphalt and the waterproofing membrane shall be removed from within this area to expose the concrete surface. The condition of the waterproofing membrane and protection board removed shall be observed and documented. A sufficient number of transverse and longitudinal bars need to be identified within each of these areas

such that at least two intersections of transverse and longitudinal bars are located. At one of the two intersections, a core shall be collected with the reinforcement bar intersection centered within the core. The length of each core is to be 150mm. After extracting the core, the exposed bar ends remaining in the bridge deck shall be coated with a waterproofing material. To minimize additional damage to the waterproofing membrane, the RCPT cores shall also be collected from within the 500mm x 500mm area exposed for ECR cores.

The 500mm x 500mm excavation may not be necessary for cores that do not contain ECR (non-ECR cores). Such cores shall be drilled to include a 150mm depth of concrete.

In the removed asphalt areas the asphalt overlay, the waterproofing membrane, and the protection board shall be collected along with the concrete sample. The inside of all core holes will be inspected before patching and any anomalies observed (such as delamination or cracking) shall be documented. The collected cores shall be packaged properly (such that any moisture is not lost and the cores do not break during shipment) and shipped to the laboratory for testing.

An equal number of cores of each type shall be collected from each curb. If deterioration of the asphalt overlay or other signs of deterioration are observed along the curb, cores for that curb shall be collected from each of the deteriorated areas first. The remaining cores for that curb lane shall be randomly located. All cores along the crown of the deck shall be randomly located. If a traffic divider is located along the crown of the deck then the required cores shall be collected along the side of the divider as close as possible to the crown of the deck. All traffic lane cores shall be randomly located. The location of all random cores shall be obtained by the use of deck grids prescribed by the Structure Rehabilitation Manual and random number generators. A flowchart of the activities for the above sample collection procedure is presented in Figure 1.

The laboratory evaluation of ECR cores shall include: cover measurement, chloride concentration profile analysis, and assessment of the quality and the condition of the epoxy coating. Laboratory testing on the non-ECR cores shall be limited to chloride concentration analyses except those specifically collected for permeability testing. If the air void system of the concrete is suspect or the concrete strength information is required, a maximum of three 100mm x 150mm cores can be collected for these purpose. This core should be taken from an area excavated for other cores to avoid additional damage to the waterproofing membrane.

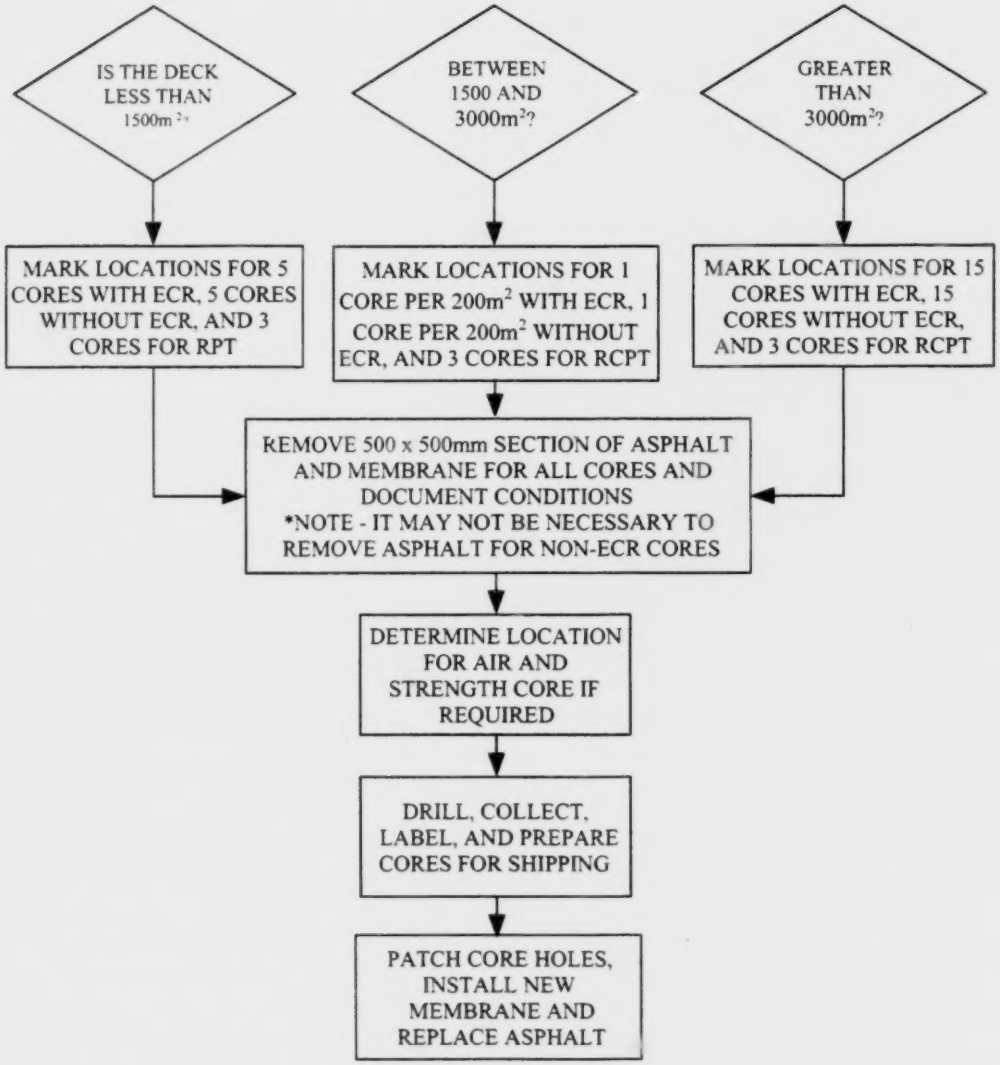

IS THE DECK LESS THAN 1500m²?
BETWEEN 1500 AND 3000m²?
GREATER THAN 3000m²?
MARK LOCATIONS FOR 5 CORES WITH ECR, 5 CORES WITHOUT ECR, AND 3 CORES FOR RPT
MARK LOCATIONS FOR 1 CORE PER 200m² WITH ECR, 1 CORE PER 200m² WITHOUT ECR, AND 3 CORES FOR RCPT
MARK LOCATIONS FOR 15 CORES WITH ECR, 15 CORES WITHOUT ECR, AND 3 CORES FOR RCPT
REMOVE 500 x 500mm SECTION OF ASPHALT AND MEMBRANE FOR ALL CORES AND DOCUMENT CONDITIONS
*NOTE - IT MAY NOT BE NECESSARY TO REMOVE ASPHALT FOR NON-ECR CORES
DETERMINE LOCATION FOR AIR AND STRENGTH CORE IF REQUIRED
DRILL, COLLECT, LABEL, AND PREPARE CORES FOR SHIPPING
PATCH CORE HOLES, INSTALL NEW MEMBRANE AND REPLACE ASPHALT

Figure 1: Phase 1 – Sampling Procedures, Bridge Deck.

Phase II

To accomplish the goals of Phase II, field testing and collection of cores for evaluation in the laboratory are required. To carry out field testing including delamination survey, cover distribution, electrical continuity testing, and half-cell potential survey substantial areas of asphalt and membrane (sawn samples) need to be removed to access the concrete surface. Cores shall be collected in sawn areas as described below. The total number of sawn samples shall be in accordance with Table 3. For locating sawn areas, every 50m length of each traffic lane and shoulder shall be considered a separate sampling area. Within each sampling area, a square sawn area shall be made having the width of the lane as one dimension. Thus a typical sawn sample area could vary between 13.7 m^2 (3.7m x 3.7m) and 24.0 m^2 (4.9m x 4.9m). When possible, up to 50% of the sawn areas shall be located in areas exhibiting deterioration within the asphalt surface.

Table 3: Sampling Frequency For Sawn Samples and Core Samples

Deck Area (m^2)	Frequency	Minimum		Maximum	
		Sawn Samples	Core Samples	Sawn Samples	Core Samples
< 1500	1 every 50m length of traffic lanes or shoulders	5	5	10	10
1500 – 3000		7	10	15	20
> 3000		12	15	15	30

In each sawn area, the asphalt overlay and waterproofing membrane shall be removed from the surface of the concrete. A visual and a delamination survey shall be performed on the exposed concrete surface. The measured area of delamination within a known sawn area can be used to extrapolate an estimate of the percentage of delaminated area on the entire bridge deck. This extrapolation is possible since more than 7% of the deck area will be exposed during the Phase II survey. All visually observable anomalies and delaminated areas are to be marked on the concrete surface and documented.

Within each sawn area, four transverse and four longitudinal ECR's shall be located and exposed by drilling 19mm diameter hole for each ECR. The clear concrete cover shall be documented on each exposed ECR including those that are exposed in spalled areas. Electrical continuity testing shall be performed using the DC technique between all exposed bars in the sawn area. If all test locations exhibit discontinuity, change the stationary ground and repeat the process. This eliminates the erroneous conclusion that all bars are discontinuous when only the stationary bar is discontinuous to all other ECR. Half-cell potentials along each exposed bar shall be performed on an interval of 150mm using the exposed area of the bar for ground connection. Two cores shall be extracted in each sawn

area, one extracting the section of the bar exhibiting the most positive potential and the other the most negative potential. If the most negative and/or the most positive potential locations happen to fall on an intersection of longitudinal and transverse bars, then the core is to be collected at the intersection extracting both ECR's. When the number of sawn areas is less than that required to collect the minimum number of cores, more than 2 cores shall be collected in the appropriate number of sawn areas to meet the minimum. Similarly no cores shall be collected from certain sawn areas if the number of sawn areas is larger than that

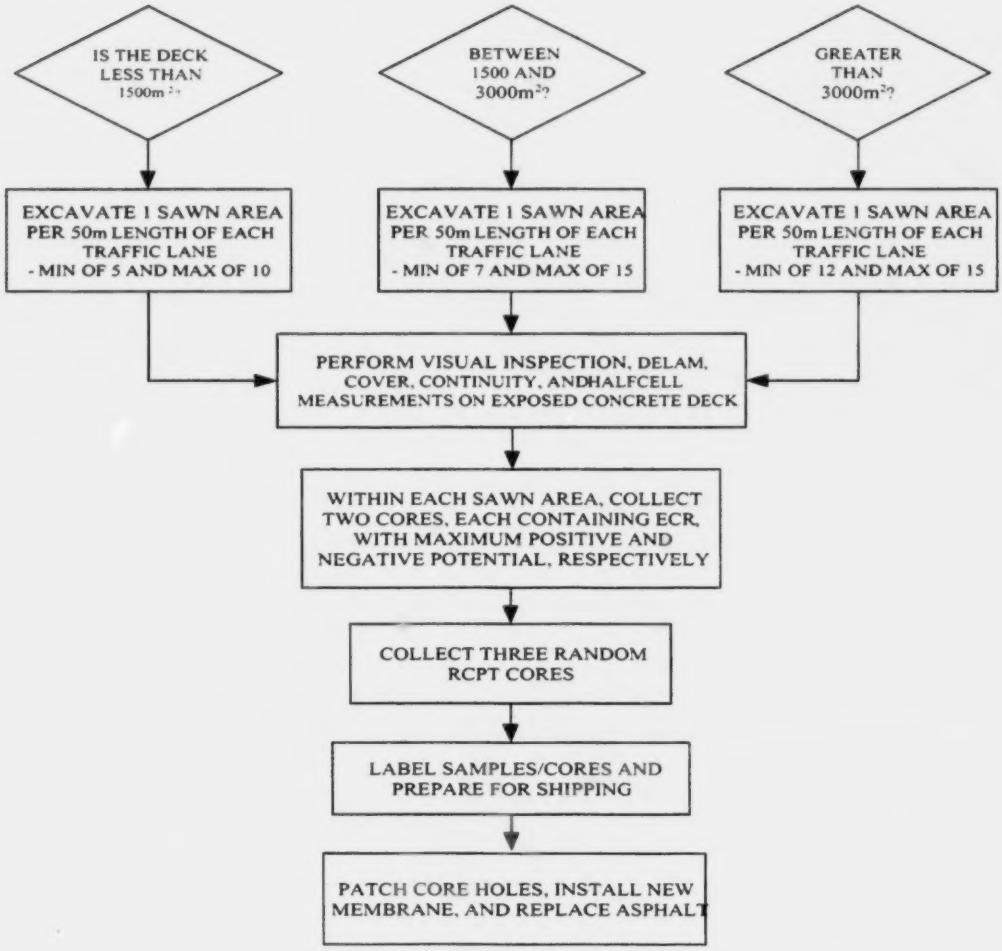

IS THE DECK LESS THAN 1500m²?
BETWEEN 1500 AND 3000m²?
GREATER THAN 3000m²?
EXCAVATE 1 SAWN AREA PER 50m LENGTH OF EACH TRAFFIC LANE - MIN OF 5 AND MAX OF 10
EXCAVATE 1 SAWN AREA PER 50m LENGTH OF EACH TRAFFIC LANE - MIN OF 7 AND MAX OF 15
EXCAVATE 1 SAWN AREA PER 50m LENGTH OF EACH TRAFFIC LANE - MIN OF 12 AND MAX OF 15
PERFORM VISUAL INSPECTION, DELAM, COVER, CONTINUITY, ANDHALFCELL MEASUREMENTS ON EXPOSED CONCRETE DECK
WITHIN EACH SAWN AREA, COLLECT TWO CORES, EACH CONTAINING ECR, WITH MAXIMUM POSITIVE AND NEGATIVE POTENTIAL, RESPECTIVELY
COLLECT THREE RANDOM RCPT CORES
LABEL SAMPLES/CORES AND PREPARE FOR SHIPPING
PATCH CORE HOLES, INSTALL NEW MEMBRANE, AND REPLACE ASPHALT

Figure 2: Phase II – Sampling Procedures, Bridge Decks.

required to collect the maximum number of cores. The extracted cores will have a diameter of 100mm and a minimum depth of 150mm and shall be extracted from sound concrete areas. The cores are to be properly labeled and packed for shipment to a laboratory. A total of three 100mm diameter cores shall be collected from three different sawn areas for rapid chloride permeability testing. The cores for rapid chloride permeability testing shall not contain any steel.

As the service life model developed in this project is not validated on actual structures, it is recommended that conventional analysis of data be employed in conjunction with the results of the service life model. The sample calculations in Appendix C show typical test results that have been plugged into the model to provide examples of the output. The results of the half-cell potential survey, electrical continuity testing, observed coating condition of extracted ECR should be used to assist in the selection of the optimal repair and protection system. When carrying out half-cell potential testing the copper-copper sulfate reference cell should be used to report all results.

The size of the sawn areas is such that the quantity of damaged concrete beneath the asphalt riding surface and the waterproofing membrane can be accurately estimated. The recommended sampling frequency for the sawn areas will remove approximately 7 to 8% of the asphalt and membrane for a typical bridge deck. While a smaller sampling area may be used, there may be an accompanying decrease in the accuracy of the damage quantity estimates. If the Phase II survey triggers a rehabilitation project the quantities estimated from the 7 to 8% surface review can be confirmed by a standard visual and delamination survey after the deck is exposed.

In this phase the condition evaluation requires a significant amount of destructive evaluation and shall be performed two years prior to a scheduled repair project. A flowchart detailing the sample collection procedure is given in Figure 2.

The removal, disposal and replacement of large quantities of asphalt and waterproofing membrane will require the services of a qualified pavement contractor for a Phase II condition survey. Such work will require traffic lane closures for a longer period of time than current procedures.

5.4.2 Barrier Walls

Review of barrier walls will only be carried out when sufficient visual damage is noted and a repair strategy is required. In the case of staged work on the structure the survey may also have to be staged.

As the exterior concrete surfaces of the barrier walls are accessible the quantity of concrete damage can be estimated by the use of sounding techniques. Information pertaining to chloride ion concentration, present condition and quality of epoxy-coated rebars, and future propensity for damage will be obtained from cores. To reduce the number of cores required,

half-cell potential measurements can be used to detect deterioration in the coating along select bars.

Visual and delamination surveys shall be performed on all three surfaces (two vertical faces and the top horizontal surface) of the barrier wall. All other testing shall be performed on the vertical surface facing the roadway, as the deicing exposure is more severe on this face. In select spans, three vertical and three horizontal ECR's shall be located on the roadside face of the barrier wall. Evenly distribute the three vertical bars chosen along the entire length of the barrier wall in each selected span. The three horizontal bars should be located near one end of the barrier wall in each span but evenly spaced along the entire height of the barrier wall.

One end of each of the three vertical bars shall be exposed in a 19mm diameter hole drilled near the top of the barrier wall. Cover measurements and any anomalies observed in each drilled hole shall be documented. Half-cell potential measurements shall be performed along each of the vertical bars at 150mm intervals over a 3m length. One end of the three horizontal bars shall also be exposed in a 19mm diameter hole. Continuing along the same horizontal bar, another hole shall be drilled at a horizontal distance of 3m. Electrical continuity between the two exposed portions of the horizontal bar shall be performed to verify that the same bar runs between each drilled hole. If continuity is not detected, another drilled hole at a horizontal distance of 3m along the same bar shall be made and continuity shall be checked between these last two drilled holes. This procedure shall be repeated until continuity is established between two consecutive drilled holes. Cover measurements and any anomalies observed in each drilled hole shall be documented. Half-cell potential measurements shall be performed at 150mm intervals along the 3m length of each horizontal bar where continuity has been determined.

A total of 6 cores (3 per side) containing ECR shall be collected from the barrier wall along the entire length of the structure. Each core shall have a diameter of 100mm and a length of 150mm. Three of these cores shall be located at sections of the ECR exhibiting the three most negative half-cell potentials and the remaining three shall be in sections exhibiting the three most positive half-cell potentials. No two cores shall be collected from the same ECR. If more than one core sampling location falls on the same ECR, the core shall be collected from another ECR exhibiting similar potential. Three additional cores shall be collected from the barrier wall in a vertical plane at elevations of 230, 460, and 690mm above the roadway. These three additional cores shall not include any ECR and will be utilized for chloride concentration analysis only. The first 6 ECR cores shall be utilized for cover, chloride distribution, and ECR evaluations. The results obtained from these cores will be used in determining the remaining service life of this element, as discussed in Part 6 of this report.

5.4.3 Abutments

Testing on the abutments will be conducted in a similar manner to that performed on the barrier walls. On the exposed surface of the abutment, a delamination survey shall be

performed and 3 vertical and 3 horizontal ECRs will be located. The ECRs shall be evenly distributed over the width and height of the abutment. On each ECR, a 3 m length is selected where both ends of the selected bar are exposed in a 19mm diameter drilled hole. The two exposed ends of each selected length will allow the bar to be evaluated for electrical continuity. If continuity is not detected than another length along the same bar shall be selected such that one of the previously exposed ends is included. Continuity between the new exposed end and the previous one shall be verified. It is essential that the length of ECR selected for testing is continuous. Electrical continuity between the identified horizontal and vertical ECR shall be performed. In addition, the half-cell potential survey will be performed on each select length at 150mm intervals from one of the exposed ends. Two cores shall be collected from sections of the ECR exhibiting the two most negative potentials and two from sections exhibiting the two most positive potentials. The extracted cores will contain the located ECR with no two cores being collected from the same select length of ECR. If any combination of the most negative and positive potentials falls on the same select length, than one of the cores shall be collected from another select length with similar potentials. In addition, three cores for chloride concentration analysis will be collected from a select vertical plane at three elevations evenly distributed over the height of the abutment. The results obtained from these cores will be used in determining the remaining service life of this element, as discussed in Part 6 of this report.

5.5 Laboratory Evaluation

All cores received at the laboratory shall be stored in a freezer (below -10°C) if they are not to be tested within 24 hours of arrival. Prior to testing, these stored cores shall be thawed for a minimum of 24 hours or until they reach room temperature (25°C). The freezer storage process will ensure that the coating, moisture content in blisters, and the coating bond remain unchanged from their initial condition at the time of extraction. In addition, freezing the sample will ensure that the chlorides do not continue to migrate.

At the onset of laboratory testing, the overall condition and the dimensions of the ECR cores shall be documented. This is to be followed by the measurement and documentation of the clear concrete cover over the top most ECR in the core. The epoxy-coated rebar segments shall then be extracted and tested as follows:

1. Visual rating of the corrosion condition of the coating using the rating scheme listed in Table 4. If the condition rating is equal to or greater than 4, then test items 2 to 4 do not have to be performed.
2. Documentation of the count of visually observable damage on the coating
3. Measurement of the number of holidays on the coating in accordance with "Standard Test Method for Holiday Detection in Pipeline Coatings," ASTM 62-87, Method A with some modifications (see Appendix A for modifications)
4. Coating adhesion testing in accordance with the standard test method "Evaluation of Internal Plastic Coatings for Corrosion of Tubular Goods by Autoclave Testing,"

NACE TM0185-93, section 5.3.2 with some modifications (see Appendix A for modifications)

Table 4: Visual Condition Rating System

CR Rating	Description
1	No evidence of corrosion.
2	A number of small, countable corrosion spots.
3	Corrosion area less than 20% of total ECR surface area.
4	Corrosion area between 20% to 60% of total ECR surface area
5	Corrosion area greater than 60% of total ECR surface area.

All evaluations pertaining to the epoxy coating shall be completed within 24 hours of the core reaching room temperature.

Chloride concentration analyses shall be performed on cores with and without ECR. For cores containing ECR, powdered concrete samples shall be collected from the section of the core above the ECR. For cores without ECR, powdered concrete samples shall be collected to the depth of 50mm. The samples are to be collected by powdering concrete from 10mm-thick semi-disks. Prior to powdering, the target section of the core will be cut into equal halves along the long axis of the core. On one of the cores halves, measurement of the 10mm semi-disks will proceed from the top of the core section. On the other core half, measurement of the 10mm semi-disks will proceed 5mm from the top. Following this procedure, the powdered samples from one core half-sample will be offset by 5mm from the powdered samples obtained from the other core half-sample. Powdered concrete samples for chloride ion content at steel depth shall be collected from 10mm disks located just above the ECR and for the background chlorides powdered concrete samples shall be collected from ECR cores in a 10 mm thick disk located at a depth of 125 and 135mm from the top of the core. The powdered samples shall be analyzed for total chloride ion content in accordance with MTO procedures. The rapid chloride permeability test shall be performed on the RCPT cores collected in Phase I or II in accordance with standard test method "Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration," AASHTO T 277-93. The uppermost 20mm of each RCPT core shall be removed and the next 100mm of the core shall be used for permeability testing (2 tests). This will minimize the impact of chlorides ions on the results of the RCPT tests. Air void analysis and compressive strength testing shall be performed in accordance with MTO standard practice.

Tables 5, 6 and 7 list the number of samples to be tested for bridge decks and for barrier walls and abutments.

Table 5: Number of Samples to be Tested in Phase I – Bridge Decks

Deck Area (m^2)	# of ECR Cores	# of Non-ECR Cores	# of Cover Measurements	# of ECR Evaluations	# CL Profiles	# of RCPT Cores
<1500	5	5	10	5	10	3
1500 to 3000	7 to 15	7 to 15	14 to 30	7 to 15	14 to 30	3
>3000	15	15	30	15	30	3

Table 6: Number of Samples to be Tested in Phase II – Bridge Decks

Deck Area (m^2)	# of ECR Cores	# of Cover Measurements	# of ECR Evaluations	# CL Profiles	# of RCPT Cores
1500	5	5	5	5	3
1500 to 3000	7 to 15	7 to 15	7 to 15	7 to 15	3
>3000	15	15	15	15	3

Table 7: Number of Samples to be Tested for Other Elements

Element	# of ECR Cores	# of Non-ECR Cores	# of Cover Measurements	# of ECR Evaluations	# CL Profiles	# of RCPT Cores
Barrier Wall	6	3	9	6	9	0
Abutment	4	3	7	4	7	0

A flowchart detailing the activities for laboratory analysis is presented in Figure 3.

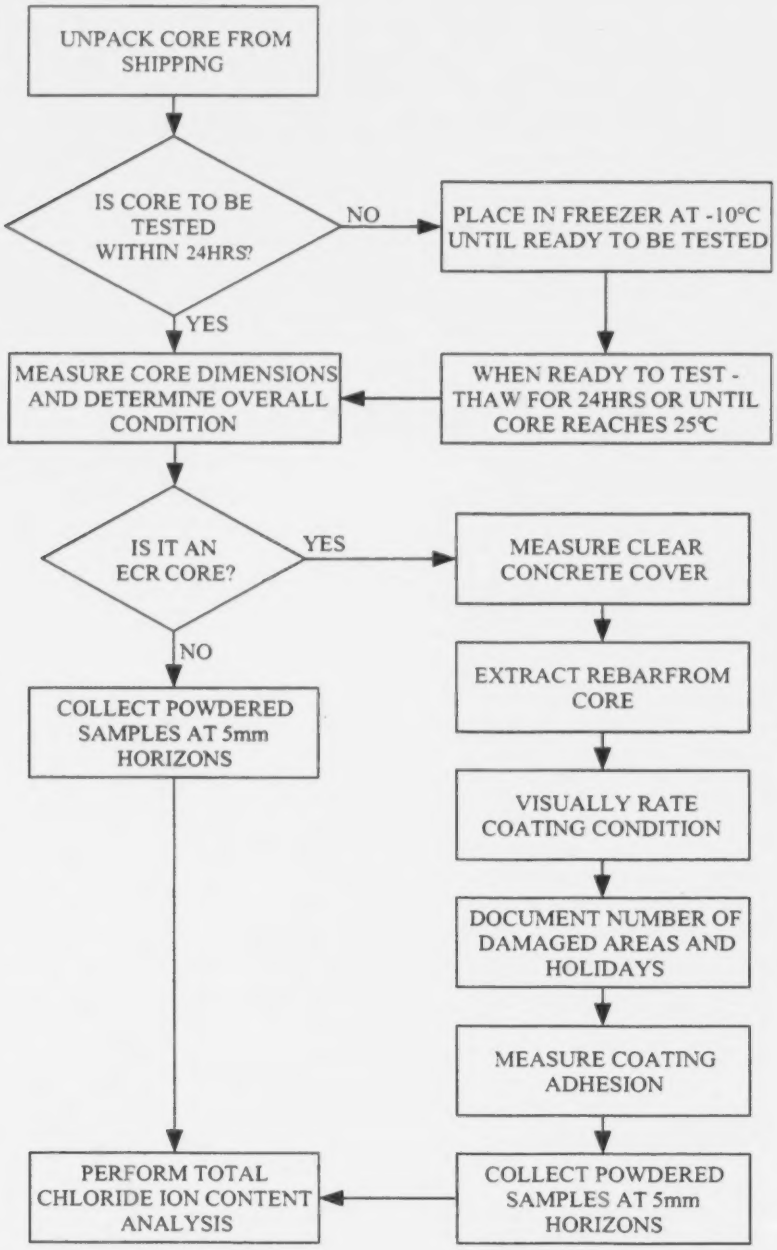

UNPACK CORE FROM SHIPPING
IS CORE TO BE TESTED WITHIN 24HRS?
NO
PLACE IN FREEZER AT -10°C UNTIL READY TO BE TESTED
YES
MEASURE CORE DIMENSIONS AND DETERMINE OVERALL CONDITION
WHEN READY TO TEST - THAW FOR 24HRS OR UNTIL CORE REACHES 25°C
IS IT AN ECR CORE?
YES
MEASURE CLEAR CONCRETE COVER
NO
EXTRACT REBARFROM CORE
COLLECT POWDERED SAMPLES AT 5mm HORIZONS
VISUALLY RATE COATING CONDITION
DOCUMENT NUMBER OF DAMAGED AREAS AND HOLIDAYS
MEASURE COATING ADHESION
PERFORM TOTAL CHLORIDE ION CONTENT ANALYSIS
COLLECT POWDERED SAMPLES AT 5mm HORIZONS

Figure 3: Laboratory Analysis Procedure for ECR and non-ECR Cores.

6. Service Life Predictions

The results of the field and laboratory evaluation provide the necessary data to estimate the time to damage on the structure. Time to damage (T_d) is defined as the age of the concrete element at which it is expected to exhibit corrosion induced damage. As discussed earlier, time to damage is a sum of time to corrosion initiation and time for propagation and can be mathematically represented as follows:

$$T_d = T_i + T_p \qquad \text{Eqn.1}$$

Where T_d is time to damage, T_i is time to corrosion initiation, and T_p is time for propagation. T_i can be estimated from chloride distribution profiles developed from cores, clear concrete cover measurements, and the results of the rapid chloride permeability test, whereas T_p can be calculated from the results of the evaluation of the epoxy-coated rebars extracted in the cores. As described later the values used in the analysis for depth of penetration of the threshold chloride concentration and the clear concrete cover can be varied. For the purpose of calculating the initiation period it is proposed that the depth of chloride penetration is selected such that the threshold concentration would only exceed the value 10% of the time. For the purpose of calculating the initiation period it is proposed that a value of concrete cover is selected such that the cover would only be lower than that value 10% of the time.

6.1 Determining the Initiation Period, T_i

A detailed discussion providing the background to the approach for predicting the initiation period is provided as Appendix B.

Most mathematical models for predicting the migration of chloride ions in concrete assume that ionic diffusion is the sole mechanism of chloride transport in concrete and use Fick's Second Law of Diffusion to predict the chloride concentration profile at any time t. This is accomplished by assuming a constant concentration of chloride ions at the concrete surface (C_s) and assigning an "apparent" diffusion coefficient (D_a) to the concrete material in the following commonly used solution of Fick's Second Law:

$$\frac{C_x}{C_s} = 1 - erf\left(\frac{x}{2\sqrt{D_a \cdot t}}\right) \qquad \text{Eqn. 2}$$

where: C_s = chloride concentration at the surface
C_x = chloride concentration at distance x and time t
x = depth from the surface
D_a = 'apparent' diffusion coefficient

As we are primarily concerned with the concentration of chloride ions at the steel depth exceeding the corrosion initiation threshold (C_t = 0.025%), the above solution for a constant C_s and D_a can be simplified to:

$$T_i = T_a \cdot \left(\frac{x_d}{x_p} \right)^2 \qquad \text{Eqn. 3}$$

where: T_i = age of the structure at initiation (years)
T_a = present age of the structure (years)
x_d = clear concrete cover (mm)
x_p = depth at which C_t has been exceeded at the time of sampling (mm)

This approach is an oversimplification and does not address the variation of surface chloride ion concentration (C_s) and permeability (or diffusivity D_a) of concrete with time. However, as the time dependent variation of C_s and D_a tend to compensate for one other, the above solution can be used to obtain a reasonable approximation of chloride profiles at any time T_a if x_p is sufficiently large. This equation is used in the service life prediction for concrete elements without surface protection (e.g. barrier walls, exposed decks).

The presence of a waterproofing membrane further complicates the prediction of chloride migration in concrete. The membrane controls the exposure of the concrete surface to the chloride ions and therefore, knowledge of its effectiveness and variation of its effectiveness with time is required to model chloride migration in such decks. To better understand the migration of chloride ions in the presence of a waterproofing membrane, the model Conflux developed by the University of Toronto was used. The results of this modeling suggested that once the membrane breaks down and significant penetration of chloride ions have occurred, T_i can be estimated using the following solution:

$$T_i = \left(\frac{x_d - x_p}{k} + \sqrt{T_a} \right)^2 \qquad \text{Eqn. 4}$$

where T_i = age of the structure when corrosion initiates (years)
T_a = age of the structure at time of sampling (years)
x_d = depth of cover (mm)
x_p = depth of threshold, C_t, at T_a (mm) determined from chloride profile of the core
RCPT = results of rapid chloride permeability test (coulombs)
$k = 0.05 \cdot RCPT$
C_t = threshold chloride ion content (0.025% by mass)

This solution requires a measure of the concrete permeability, which will be determined indirectly by conducting the "rapid chloride permeability test" (ASTM C 1260) on several cores. A more detailed discussion on the development of the model can be found in Appendix B.

For each core, T_i can be estimated by extracting x_p from the chloride ion concentration profile of the core established by analysis of powdered concrete samples from 5mm horizons and estimating the value of k from the rapid chloride permeability test results. The value of x_d is the clear concrete cover over the topmost bar in each core. A flow chart summarizing the procedure for calculating T_i is shown in Figure 4.

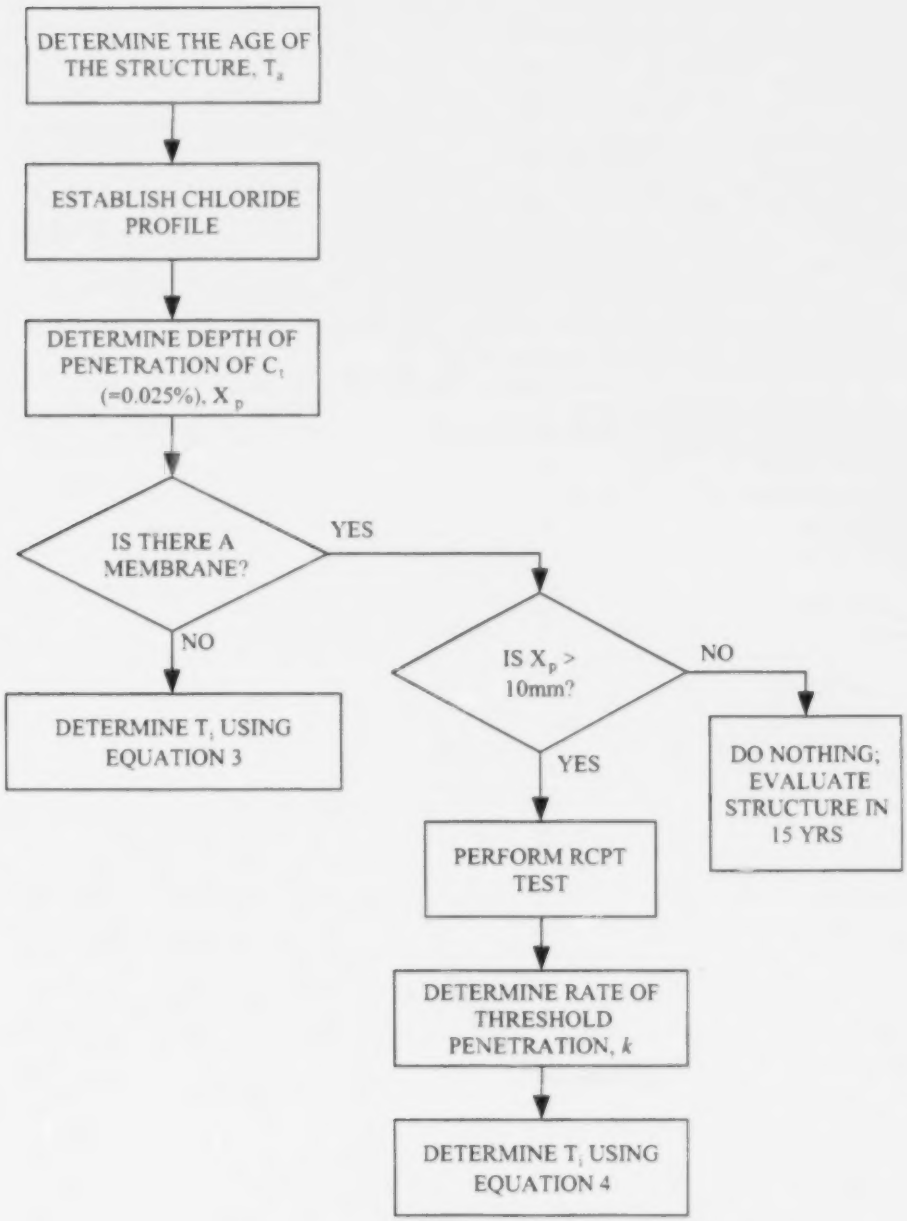

DETERMINE THE AGE OF THE STRUCTURE, T_a
ESTABLISH CHLORIDE PROFILE
DETERMINE DEPTH OF PENETRATION OF C_t (=0.025%), X_p
IS THERE A MEMBRANE?
YES
NO
IS X_p > 10mm?
NO
YES
DETERMINE T_i USING EQUATION 3
DO NOTHING; EVALUATE STRUCTURE IN 15 YRS
PERFORM RCPT TEST
DETERMINE RATE OF THRESHOLD PENETRATION, k
DETERMINE T_i USING EQUATION 4

Figure 4: Procedure for Calculating Corrosion Initiation Time (T_i)

6.2 Determining the Propagation Period, T_p

The primary goal of evaluating the condition of the epoxy coating is to estimate the time to propagation and therefore, some mechanism is required to relate the quality of the coating to T_p (time to propagation). Two recent studies, one by the National Cooperative Highway Research Program (NCHRP) [42] and the other by the Florida Department of Transportation[43] have linked time to propagation to the quality of the coating. The NCHRP study evaluated time to propagation for epoxy coated rebars from various sources in laboratory specimens exposed to two different conditions, natural weather and salt-water ponding. This study concluded that the epoxy coating deteriorates with time and the rate of deterioration is dependent on the initial number of defects on the coating and the exposure environment. The study also developed mathematical relationships to describe time to propagation as a function of coating quality described by the number of defects per unit length. It further observed that when the number of coating defects exceeds 40 defects per meter (threshold value) and sufficient chloride ions are present to initiate corrosion, the coating does not provide any protection against corrosion and the cumulative flow of the corrosion current determines the remaining time to damage. The Florida study found a reasonable agreement between the rate of deterioration observed on marine piles of 5 major bridges in the state and the results of a mathematical model that assumed time to propagation to be a function of the quality of the coating as described by the number of defects. The results of the service life models developed under NCHRP study also agree well with the Florida ECR experience in marine pilings.

The formula developed by the NCHRP study can be used to estimate time to propagation T_p for each epoxy coated rebar extracted in a core as follows:

$$T_p = T_1 + T_2 \qquad \text{Eqn. 5}$$

T_1 represents the time for defects to exceed the value of 40 defects per metre after the chloride ion concentration has exceeded the threshold value. T_2 represents the time required for damage to occur after both the chloride ion concentration and the number of defects have exceeded their threshold values.

6.2.1 Bridge Decks and Abutments

Of the two exposure conditions evaluated by the NCHRP study, the natural weathering can be equated to the Ontario's exposure conditions for bridge decks and abutments and T_1 for each rebar extracted from a core can be estimated as follows:

$T_1=2.5(40/N_i-1)$ for $N_i < 40$ Eqn. 6a

$T_1=0$ for $N_i => 40$ Eqn. 6b

$T_1=0$ for CR=> 4 Eqn. 6c

For the development of this equation it was assumed that N_i is the number of defects per metre at the time of the survey and not at the time of construction. The quantity CR is the

coating condition rating as described in Table 4. A reduction in adhesion reduction can occur in areas that are not adjacent to visually observable damage [38]. If holidays are present in the area of adhesion reduction, corrosion can initiate under the coating due to migration of chloride ions through the holidays. To account for reduction in time to damage from this effect the following modification to Eqn. 6 was developed:

$$T_1 = T_1 (-0.05AR + 1.05) \qquad \text{Eqn. 7}$$

where AR is the average Adhesion Rating of the bar on a scale of 1 to 5. This modification assumes that T_1 is reduced by 10% for an adhesion rating of 3 (some loss of adhesion) and by 20% for an adhesion rating of 5 (complete loss of adhesion). There is no support for this modification in literature and this may need to be revised based on experience in Ontario.

T_2 was calculated to be 16 years for natural weathering by the NCHRP study using a cumulative corrosion charge transfer of 2 _A-year. This means that once the chloride ion concentration and the number of defects exceeded their thresholds, it would take a maximum of 16 years before concrete damage would result. Therefore, if the cumulative corrosion current or charge transfer to date could be measured, then time to concrete damage could be estimated. The rate of charge transfer is a function of the number of defects N_i and sufficient information is not available in the NCHRP report to develop a relationship between T_2 and N_i. To develop this relationship, average number of defects and average condition rating were analyzed for 473 ECR collected from 80 bridge decks in the State of New York and Pennsylvania. The number of defects on these ECR averaged 130 per meter (including holidays) for an average condition rating of 1.1. Based on this information, it was assumed that until the ECR developed more than 40 defects per meter, T_2 can be assigned the maximum value of 16 years and for defects exceeding 200 per meter, it can be assigned the minimum value of 3. Based on these assumptions the following relationship was developed:

$T_2 = 16$	for $N_i < 40$	Eqn. 8a
$T_2 = 19.25 - 13N_i/160$	for $40 \leq N_i < 200$	Eqn. 8b
$T_2 = 3$	for $N_i \geq 200$	Eqn. 8c

When half-cell potential data is available, the propagation time T_p for a section of the bar represented by the half-cell potential is to be calculated as described previously with the following modifications:

- Half-cell potential data will be used to obtain additional T_1 (time to threshold coating defects) values. All 150mm sections of ECR exhibiting -350mV (versus Cu-$CuSO_4$)

are assigned the T_I value calculated for the core extracted from the section of the bar with most negative potential. If more than one core has been extracted then the average T_I shall be assigned. Similarly, all 150mm sections of ECR that exhibited half-cell potential more positive than -350mV are assigned the T_I value calculated for the core extracted from the section of the bar with the most positive potential.

It should be noted that the propagation model is based on results of one study and it is recommended that MTO develop a data base to ascertain what the deterioration of the epoxy in Ontario is and thus better define equations 6 to 8.

6.2.2 Barrier Walls

Due to direct exposure to chloride ions the NCHRP salt water ponding exposure data was considered appropriate for the barrier walls.

For salt water ponding exposure, the following formulation to predict time required for the number of defects to exceed the threshold value for ECR elements was developed by the NCHRP study:

$T_I=0.37(40/N_i-1)$ for $N_i < 40$ Eqn. 9a
$T_I=0$ for $N_i => 40$ Eqn. 9b

where T_I is time in years required to reach the threshold value. It has been assumed that the number of defects at the time of construction can be replaced by Ni, the number of defects at the time of the survey.

Equation 6a was modified for use in Ontario to account for the reduction in adhesion as follows:

$$T_I=T_I\,(-0.05AR+1.05) \qquad \text{Eqn. 10}$$

where AR is Adhesion Rating on a scale of 1 to 5.

This modification reduces T_I by 10% for reduction in adhesion, an adhesion rating of 3 and reduces it by 20% for complete loss of adhesion, an adhesion rating of 5.
The NCHRP study suggests that the maximum time to damage for salt-water ponding is 2 years. To account for the colder temperatures in winter, lower humidity, and non-constant exposure to chlorides, the maximum time to damage for Ontario was assumed to be 8 years and a minimum of 3 years. The following equation was developed to calculate T_2 as a function of the number of defects such that for N_i equal to or less than 40 results in the maximum time to damage of 8 years and N_i greater than 200 results in the minimum time to damage of 3 years.

$T_2 = 8$ for $N_i < 40$ Eqn. 11a
$T_2 = 9.25 - N_i/32$ for $40 \leq N_i < 200$ Eqn. 11b
$T_2 = 3$ for $N_i \geq 200$ Eqn. 11c

A flow chart summarizing the procedure for calculating T_p is shown in Figure 5.

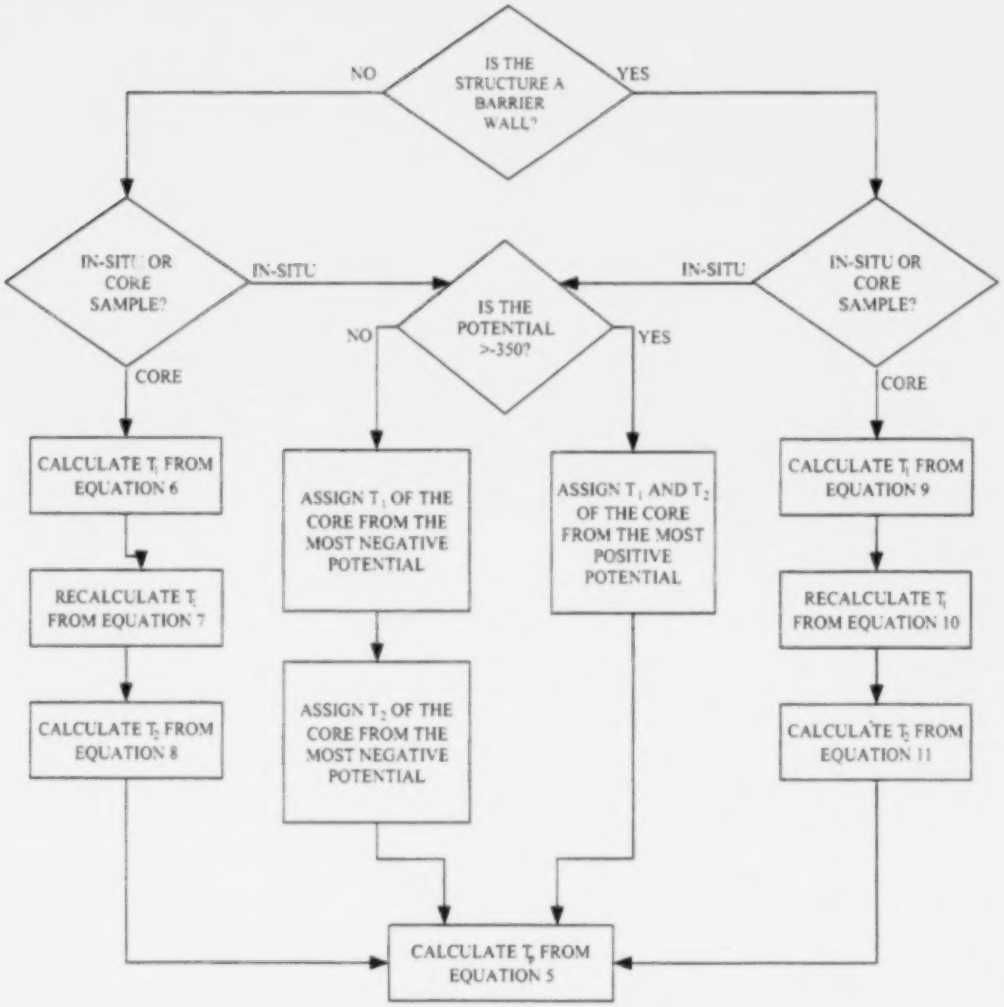

IS THE STRUCTURE A BARRIER WALL?
NO
YES
IN-SITU OR CORE SAMPLE?
IN-SITU
IS THE POTENTIAL >-350?
IN-SITU
IN-SITU OR CORE SAMPLE?
NO
YES
CORE
CORE
CALCULATE T1 FROM EQUATION 6
ASSIGN T1 OF THE CORE FROM THE MOST NEGATIVE POTENTIAL
ASSIGN T1 AND T2 OF THE CORE FROM THE MOST POSITIVE POTENTIAL
CALCULATE T1 FROM EQUATION 9
RECALCULATE T1 FROM EQUATION 7
RECALCULATE T1 FROM EQUATION 10
CALCULATE T2 FROM EQUATION 8
ASSIGN T2 OF THE CORE FROM THE MOST NEGATIVE POTENTIAL
CALCULATE T2 FROM EQUATION 11
CALCULATE Tp FROM EQUATION 5

Figure 5: Procedure for Calculating Time for Propagation (T_p)

6.3 Summary

The preceding discussion outlines approaches for predicting the initiation period, T_i, and the propagation period, T_p, the sum of the two being the estimated age, T_d, at which the structure exhibits significant damage as follows:

$$T_d = T_i + T_p \qquad \text{Eqn.1}$$

For bridge decks with protective waterproofing membranes the time to damage is given by the following equation:

For $N_i < 40$

$$T_d = \left[\left(\frac{x_d - x_p}{k} + \sqrt{T_a}\right)^2\right] + \left[2.5 \cdot \left(\frac{40}{N_i} - 1\right) \cdot (1.05 - 0.05AR)\right] + [16] \qquad \text{Eqn.12a}$$

For $40 \le N_i < 200$

$$T_d = \left[\left(\frac{x_d - x_p}{k} + \sqrt{T_a}\right)^2\right] + [0] + \left[19.25 - \frac{13N_i}{160}\right] \qquad \text{Eqn.12b}$$

For $N_i \ge 200$

$$T_d = \left[\left(\frac{x_d - x_p}{k} + \sqrt{T_a}\right)^2\right] + [0] + [3] \qquad \text{Eqn.12c}$$

For unprotected elements such as barrier walls or exposed bridge decks the equation for predicting the time to damage is:

For $N_i < 40$

$$T_d = \left[T_a \cdot \left(\frac{x_d}{x_p}\right)^2\right] + \left[0.37 \cdot \left(\frac{40}{N_i} - 1\right) \cdot (1.05 - 0.05AR)\right] + [8] \qquad \text{Eqn.13a}$$

For $40 \le N_i < 200$

$$T_d = \left[T_a \cdot \left(\frac{x_d}{x_p}\right)^2\right] + [0] + \left[9.25 - \frac{N_i}{32}\right] \qquad \text{Eqn.13b}$$

For $N_i \ge 200$

$$T_d = \left[T_a \cdot \left(\frac{x_d}{x_p}\right)^2\right] + [0] + [3] \qquad \text{Eqn.13c}$$

For abutments the equation for predicting time to damage is:

For $N_i < 40$

$$T_d = \left[T_a \cdot \left(\frac{x_d}{x_p}\right)^2\right] + \left[2.5 \cdot \left(\frac{40}{N_i} - 1\right) \cdot (1.05 - 0.05AR)\right] + [16] \qquad \text{Eqn.14a}$$

For $40 \leq N_i < 200$

$$T_d = \left[T_a \cdot \left(\frac{x_d}{x_p}\right)^2\right] + [0] + \left[19.25 - \frac{13N_i}{160}\right] \qquad \text{Eqn.14b}$$

For $N_i \geq 200$

$$T_d = \left[T_a \cdot \left(\frac{x_d}{x_p}\right)^2\right] + [0] + [3] \qquad \text{Eqn.14c}$$

Symbols

T_a = Age of structure at time of sampling (years).

T_d = Age of structure at time of significant damage (years).

T_i = Age of structure when corrosion initiates (years).

T_p = Time to propagation (years) i.e., $T_d - T_i$

x_d = Depth of cover to rebars (mm)

x_p = Depth of threshold chloride ion content (0.025% by mass)

k = 0.05 x RCPT test value (coulombs)

N_i = Number of defects per meter of ECR sample

7. Condition Survey Cost Estimates

The cost of the proposed bridge deck Phase I and Phase II Condition Surveys is compared to a typical condition survey project of a black steel reinforced deck based on the requirements of the present Structure Rehabilitation Manual. Surveys of barrier wall and abutment elements have also been upgraded and costs for them are described as well. Trow has been involved in a multitude of condition surveys since they were first introduced in the 1970's and as such has a large database of project information.

The cost of carrying out a condition survey will vary with the size of the bridge being surveyed but the unit cost (per square metre) will decrease with increasing bridge deck size. Some other aspects that can affect the cost of the survey include; traffic control, night work, travel time, etc.

7.1 Cost of Black Steel Condition Survey

For more than 30 years consultants have carried out all condition surveys for MTO programs, initially based on a consultant rotation basis but recently selection has been based on a bidding program for the work. The unit price for carrying out this work has decreased steadily over the last 10 years as more consultants entered the market and were approved for this work.

A number of recent projects carried out by Trow were reviewed to estimate the cost of carrying out this work (Table 8). The three projects chosen are of differing sizes and give a good distribution of costs. All projects are for the 2000 construction season and represent competitive bids on MTO structures.

Table 8: Bridge Deck Condition Survey, Year 2000 Costs

Project #	Deck Size (m^2)	Total Cost ($)	Unit Cost ($/m^2$)
37-342, 401/Yonge St.	177	8,142	46.00
37-326, 401/404	702	14,764	21.03
37-201, 401/Wilson Ave.	1,725	29,926	17.35

Traffic control costs are included in all three projects and the fieldwork was carried out at night. The costs shown are for the minimum number of samples specified in the Structure Rehabilitation Manual. The following table (Table 9) breaks down the cost of each project based on field costs, lab costs, traffic control costs and reporting costs.

Table 9: Bridge Deck Condition Surveys, Cost Breakdown

Project #	Size (m^2)	Field (%)	Lab (%)	Traffic Control (%)	Reporting (%)
37-342	177	41	6	26	27
37-326	702	45	9	25	21
Typical Deck	1,500	50	7	27	16
37-201	1,725	53	5	28	13

Based on this data, for a typical 1,500 m^2 deck the anticipated unit cost would be approximately $18 per square metre. This unit cost translates to a total cost of $27,000 for a typical condition survey of a black steel reinforced deck based on the present SRM. The cost breakdown is estimated at 50% fieldwork, 7% lab work, 27% traffic control and 16% for reporting.

7.2 Cost of Proposed Phase I Condition Survey of a Typical ECR Deck

The following costs (Table 10) are the estimated costs for carrying out a Phase I condition survey of a 1,500 square metre asphalt covered deck with epoxy-coated reinforcement using the proposed protocols.

Table 10: Phase I Condition Survey, Estimated Costs

Survey Item Description	Cost Implications	Cost ($)	Cost (%)
Fieldwork (including drilling) will be similar to existing	Similar cost as existing	13,500	41
Lab work including: 10 cores for chlorides (12 samples per core), 1 core for air voids and comp. strength, 5 evaluations of the epoxy coating, 3 RCP tests	120 chlorides @$60 1 air void @ $250 1 comp. str. @ $25 5 epoxy eval.@$30 3 RCP @ $250	7,175	22
Traffic Control	Similar cost as existing	7,290	22
Reporting	Additional $1,000 for service life prediction calculation and data reporting	5,320	15
Total		**33,285**	**100**

The cost for the lab work in the ECR Condition Survey has increased significantly from the existing protocol. The unit cost for a typical 1,500 m^2 deck is $22.19 per square metre. The additional chloride profiles, concrete permeability and epoxy quality are important inputs for the service life prediction equations. The total price for the ECR Condition Survey is 23% higher than the estimated cost of the existing protocol on a black steel reinforced deck. These prices may decrease as consultants become familiar with the protocol.

7.3 Cost of Proposed Phase II Condition Survey of a Typical ECR Deck

The following costs (Table 11) are the anticipated costs for carrying out a Phase II condition survey of a typical 1,500 square metre asphalt covered deck with epoxy-coated reinforcement using the proposed protocols. For the example shown it is assumed that the bridge deck surface has more than 2% damage at the time of the condition survey and hence the focus of the work is to collect data for the rehabilitation selection and the rehabilitation contract documents.

Table 11: Phase II Condition Survey, Estimated Costs

Survey Item Description	Cost Details	Cost ($)	Cost (%)
Fieldwork includes substantial additional costs for asphalt/waterproofing removal, disposal and replacement, in-situ electrical tests	Estimated $10,000 additional costs	23,500	55
Lab work includes; 5 cores for chlorides (12 samples per core), 5 evaluations of the epoxy coating, 3 RCP	60 chlorides @$50 5 epoxy eval. @ $30 3 RCP @ $250	3,900	9
Substantial additional costs for traffic control	Estimated $4,000 additional costs	11,290	26
Reporting	Similar cost as existing	4,320	10
Total		**$43,010**	**100**

The estimated cost of a Phase II condition survey, as noted above, for a typical ECR bridge deck is substantially greater than the existing protocol for a "first time" survey of a black steel deck. The premium is in the order of 59%. The additional costs are mainly due to removal, disposal and replacement of much larger quantities of asphalt, waterproofing membrane and protection board and increased traffic control costs. The need to remove large areas of asphalt pavement (approximately 7% for ECR compared to less than 0.1% for black steel) is to obtain better quality information regarding the physical condition of the deck slab for rehabilitation contract documents particularly since in most cases rebar corrosion

potential data will not be available. Asphalt sawn samples on black steel decks, typically ten 250 mm x 250 mm samples for a 1,500 m^2 deck slab are readily obtained and the relatively small cold-mix samples can be satisfactorily placed and compacted with a minimum of equipment. In comparison for an ECR deck, removing, disposing and replacing more than 100 m^2 of asphalt in a satisfactory manner is a task for an experienced paving contractor.

The cost of this work is inflated by the need to close the deck slab (or part of the deck slab) for the day following the nighttime fieldwork so that the waterproofing membrane, the protection board and the asphalt layers can be properly installed. Such a schedule requires an additional day of traffic protection.

It is clear that the cost of a Phase II condition survey will be heavily dependant on traffic control arrangements. For example, in some cases, although it may be possible to close all lanes of a bridge for the nighttime fieldwork it may not be acceptable to retain such a closure for the following day.

7.4 Barrier Wall and Abutment Review

The current SRM contains limited detailed guidance on condition survey field procedures for concrete components other than bridge decks. Where the barrier walls and/or abutments are in apparently good condition a review of the chloride profiles, concrete cover and quality of the epoxy coating is required to make an estimate of the remaining service life of these elements. The costs over and above the standard visual and delamination survey, which are included in the current protocols, are detailed below (Tables 12 and 13):

Table 12: Barrier Walls (Per Structure)

Survey Item Description	Cost Details	Cost ($)
Fieldwork includes additional coring and in-situ electrical tests	Additional core drilling - $300 Electrical tests - $500	800
Lab work including; 9 cores for chlorides (12 samples per core), 6 evaluations of the epoxy coating	108 chlorides @$50 6 epoxy eval. @ $30	5,580
Traffic Control	Already included	
Reporting	Extra $500 for service life prediction calcs.	500
Total		**$6,880**

Table 13: Abutments (Per Structure)

Survey Item Description	Cost Details	Cost ($)
Fieldwork includes additional coring	Additional core drilling $300	300
Lab work including; 7 cores for chlorides (12 samples per core), 4 evaluations of the epoxy coating	84 chlorides @$50 4 epoxy eval. @ $30	4,320
Traffic Control	Already included	
Reporting	Extra $500 for service life prediction calcs.	500
Total		**$5,120**

8. Future Work

The rapid and successful implementation of the condition survey protocol on structures constructed with ECR, by Ontario consulting engineers, the effectiveness of the testing and data interpretation, and the reliability of the prediction of the remaining service of the bridge component, is hampered by a number of factors. These include:

- Unfamiliarity with test methods not included in the current Structure Rehabilitation Manual such as epoxy coating condition rating, chloride sampling of core samples at consecutive horizons, concrete permeability tests and location of rebar intersection points in the field.

- Lack of reliable statistical data on the achieved concrete cover to reinforcing steel in structures built after 1978.

- Lack of reliable statistical data on the distribution of chlorides in the near surface of bridge decks constructed after 1978.

- Lack of data on the corrosion condition of ECR in bridge decks and substructure elements.

The training and familiarization of consultant's technical staff in the implementation of the condition survey protocol, including the new test methods, can be largely accomplished in a 2 day workshop conducted by qualified staff providing that such training is supplemented by periodic monitoring and review by MTO staff. This periodic monitoring could include an annual update course to maintain certification of staff. The procurement of reliable statistical data regarding concrete cover to steel reinforcement, chloride distribution and condition of the ECR will be an ongoing project as data on these parameters is obtained and analyzed from routine condition surveys. However confidence in, and refinement of, the proposed condition survey protocol and the service life prediction will be maximized if some data can be obtained in the short term. There are a number of options to collect this statistical data on bridge decks, in year 2002, as follows:

1) MTO to set up a demonstration project in which the proposed condition survey protocol would be carried out by experienced consulting engineers on a number of bridge decks constructed in the "early days" of ECR in Ontario. Bridges in the demonstration project should be located on main highways and secondary highways.

2) MTO review the program of bridge evaluation projects scheduled for 2002 to include a number of bridge decks constructed with ECR. The proposed condition survey protocol would be used on such bridge decks.

3) MTO arranges to collect data from any post 1978 bridge decks, which are "exposed" i.e., the bituminous pavement, protection board and waterproofing membrane are removed and replaced. Such projects may occur for bridge decks for which a condition survey has already been carried out or for bridge decks where the bituminous system is replaced for reasons other than deck slab rehabilitation, i.e., a modified "shave and pave" operation.

4) When Phase I condition surveys are carried out and the depth of chloride penetration is insignificant i.e., <10mm, the tests involving the quality and condition of the recovered ECR samples should be carried out even though such data is not required for the service life prediction.

The proposed 2 day workshop to train consulting engineers staff on the condition survey protocol and test methods will be of greater benefit if some of the bridge deck field investigations noted above have been carried out and reported prior to the training.

9. Summary and Conclusions

Based on current state-of-the-art knowledge this report documents a methodology for conducting condition surveys on structures constructed with epoxy coated reinforcement (ECR). The report also includes equations to predict the remaining service life of bridge components constructed with ECR.

There are significant differences in the proposed condition survey protocols for ECR bridge components compared to the current, long established procedures for structures built with uncoated steel. The differences apply to data collection, sampling and testing and importantly to the scheduling of a condition survey. For example the improved durability of bridge decks constructed since the mid 1970's, due to the use of waterproofing membranes, increased cover to top steel, improved quality assurance programs etc. leads to the conclusions that the first time (Phase I) condition survey would typically be carried out on a 20 year old deck unless repaving is being carried out earlier and the closure of the deck to traffic allows for coring and testing to be done. Since significant damage would not normally be anticipated at 20 years the Phase I condition survey procedures would focus on data collection to predict when corrosion induced damage would likely occur and when a Phase II condition survey would be required to support subsequent rehabilitation.

The current condition survey procedures rely heavily on the standard ASTM C876 corrosion potential test to determine the deck rehabilitation method and in the calculation of concrete removal quantities documented in the rehabilitation contract documents. Since such corrosion potential measurements do not apply to bridge components reinforced with ECR modifications to the current sampling and testing program are proposed. A significant increase in chloride testing for the concrete cover layer is necessary and tests to determine the permeability of the concrete and the condition of the ECR are included in the condition survey protocol. In a Phase II condition survey a significant increase in the size of the asphalt sawn samples is proposed to produce sufficient data regarding the condition of the concrete. Such data should lead to increased accuracy of the concrete removal quantities contained in the rehabilitation contract documents. The removal, disposal and replacement of substantial quantities of asphalt material will require the services of a pavement contractor.

The additional testing of concrete and rebar samples, since the corrosion activity of the epoxy coated reinforcement cannot be directly measured, will cause an increase in the cost of condition surveys. For a typical bridge deck it is estimated that the cost of a Phase I condition survey will be approximately 23% higher than the cost of a first time condition survey of a black steel deck carried out in accordance with the MTO Structural Rehabilitation Manual. (However the total cost of carrying out investigations and condition surveys on ECR bridge decks should not increase due to the quality improvements since the mid 1970's and the reduced frequency of condition surveys).

The formulae developed to predict the remaining service life of a bridge component, which essentially is the sum of the corrosion initiation period and the corrosion propagation period, is considered to be the best approach at this time for the MTO structure rehabilitation program. Predicting the period between construction and the onset of corrosion of the embedded steel reinforcement when little chloride has penetrated the concrete surface is not possible with the conventional approach and for such cases, since corrosion damage is not likely to occur in the short term, a more arbitrary approach is recommended.

The lack of statistical data for Ontario bridges related to achieved concrete cover to rebars, distribution of chlorides and corrosion condition of ECR increases the cost of condition surveys and reduces the reliability of remaining service life predictions. The report recommends a number of options to collect some statistical data over the short term.

Recommendations are also presented on training consultant's staff in the implementation of the condition survey protocol including the new test methods.

APPENDIX A
User's Guide to the Condition Survey Protocol, Field and Laboratory Tests

APPENDIX A

FIELD SAMPLING PROTOCOL FOR BRIDGE DECK CONDITION SURVEY - PHASE I

A. Core Sampling Frequency and Locations

1. From bridge drawings estimate the total deck area of all spans in the bridge.

2. Use Table 1 to determine the number of cores to be collected.

Table 1: Frequency of Core Sampling

Deck Area (m^2)	# of Cores with ECR	# of Cores without ECR	Rapid Chloride Permeability Cores
< 1,500	5	5	3
1,500 to 3,000	1 per 200m^2	1 per 200m^2	3
> 3,000	15	15	3

3. Distribute the location of non-RCPT cores throughout the bridge deck as described in Table 2 below.

Table 2: Sampling Locations and Distributions

Sampling Locations	Core Locations	Percent of Cores with ECR	Percent of Cores without ECR
Curb	From a 0.5 m wide strip of pavement along the curb	20%	20%
Crown of Deck	From a 0.5 m wide strip of pavement centered along the crown of the deck	10%	10%
Traffic Lanes	Within the traffic lanes	20%	20%

4. The exact core sampling locations shall be selected using by random number generators or other methods to generate the coordinates of the sampling location.

B. Core Sample Collection

1. At each ECR core location:

   a. Locate at least 2 transverse and horizontal bar intersections. If the intersections cannot be located through the asphalt overlay, excavate a 500mm x 500mm area removing the asphalt

layer and waterproofing membrane. Collect the asphalt and the waterproofing membrane for evaluation in the laboratory.

b. Extract a 100mm diameter x 150mm deep core centered at one of these intersections. The core must contain at least one ECR. If no steel is extracted in the core, another core at the other intersection shall be collected.

c. If no 500mm x 500mm excavation is made, collect the asphalt and the waterproofing membrane along with the core.

2. At each non-ECR core location:

   a. Locate steel where possible using a rebar locator and select an area for coring to avoid extracting steel in the core. If the steel reinforcement cannot be located through the asphalt overlay, excavate a 500mm x 500mm area removing the asphalt layer and waterproofing membrane. Collect the asphalt and the waterproofing membrane for evaluation in the laboratory.

   b. Extract a 100mm diameter x 150mm deep core.

   c. If no 500mm x 500mm excavation is made, collect the asphalt and the waterproofing membrane along with the core.

3. At each rapid chloride permeability core location:

   a. Select three ECR core sampling areas, expand the excavation if necessary.

   b. Locate steel to avoid extracting in the core.

   c. Collect a 100mm diameter x 150mm deep concrete core.

4. Inspect and document any anomalies (such as cracking) in the interior of all core holes.

5. Repair all core holes as per standard MTO practice. Label and prepare all cores for shipment. The cores need to be prepared such that they are not broken

during shipment and that any contained moisture is not lost.

FIELD SAMPLING PROTOCOL FOR BRIDGE DECK CONDITION SURVEY - PHASE II

A. Sampling Frequency and Location

1. Sawn areas shall be randomly located with at least one sawn area in each traffic and shoulder lane for every 50m in length of the lane.

2. Sampling frequency within sawn areas shall be in accordance with Table 3.

Table 3: Sampling Frequency For Sawn Samples and Core Samples

Deck Area (m^2)	Frequency	Minimum		Maximum	
		Sawn Samples	Core Samples	Sawn Samples	Core Samples
< 1500	1 every 50m length of traffic lanes or shoulders	5	5	10	10
1500 – 3000		7	10	15	20
> 3000		12	15	15	30

3. When possible, 50% of sawn areas shall be located in areas that exhibit deterioration of the asphalt and the waterproofing membrane.

4. For each sawn area:

   a. Excavate a square area with the width of the lane as its dimension.

   b. Remove all asphalt and waterproofing membrane.

B. Field Evaluation Within Sawn Areas

1. Perform a visual survey of the entire concrete surface after the asphalt and the waterproofing membrane have been removed. Document any anomalies observed.

2. Perform a delamination survey of the entire concrete area exposed in the sawn sample and document the

size and location of each delamination on a sketch of the sawn area. Mark the delaminated areas on the exposed concrete surface.

3. Locate 4 adjacent transverse and 4 adjacent longitudinal ECR in the sawn area using a rebar locator. Mark the entire length of each bar on the exposed concrete surface and label them.

4. Expose each ECR in a 19mm drilled hole[I] while minimizing damage to the ECR during the excavation. Remove sufficient epoxy coating from the ECR to allow electrical contact with the steel.

5. Measure and document clear concrete cover over the reinforcing steel to the nearest mm[I]. Concrete cover shall be measured from the highest point of the bar to the top of the concrete surface. If the bar is not centered in the hole, measure cover from the highest point accessible in the hole to the top of the concrete surface.

6. Perform electrical continuity testing between each exposed bar as follows:

   a. Select one ECR as the stationary ground[II].

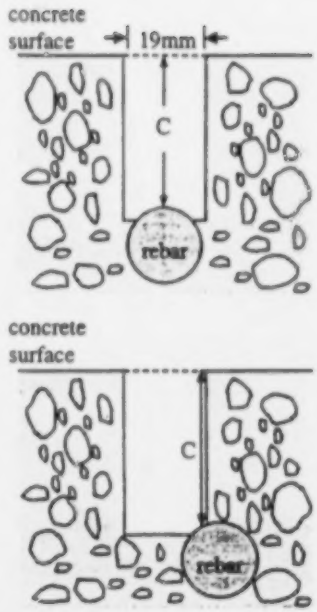
   b. Drill a hole in the ECR and install a threaded screw. Attach one terminal of an electrical multimeter (with a function to read resistance and a 10MΩ input impedance) to the screw.

   c. Attach the other terminal of the multimeter to another ECR and read the resistance. Record this reading as forward resistance.

   d. Reverse the terminal connections on the multimeter (i.e., connect the test lead from the positive terminal to the negative terminal and the test lead in the negative terminal to the positive terminal) and read the resistance again and record the reading as reverse resistance[III].

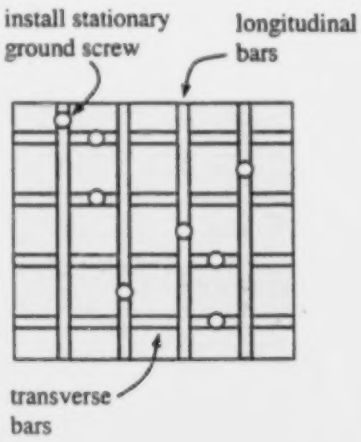
   e. Measure the voltage drop between the two ECR in mV to the nearest mV.

I

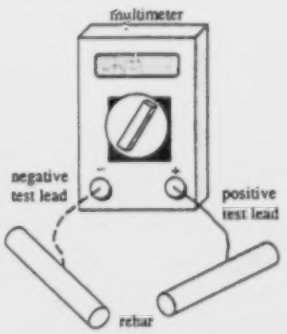


C = clear concrete cover

II



III Hook up for forward resistance measurement.



f. Perform steps c to e for each combination of the stationary ground and each of the remaining ECR located in the test area.

g. Use the following criteria to determine continuity:

   i. If both the forward and reverse resistances are less than 3Ω and the difference between them is less than 1Ω, and the mV reading is less than 1mV, the two ECR are continuous.

   ii. If the forward and the reverse resistances are larger than 3Ω or the difference between the forward and reverse resistances is more than 1Ω or the mV reading is more than 1mV, the ECR are discontinuous.

   iii. If either the forward or the reverse resistance is negative, a macrocell current is most likely flowing in the steel mat. The continuity of steel is uncertain.

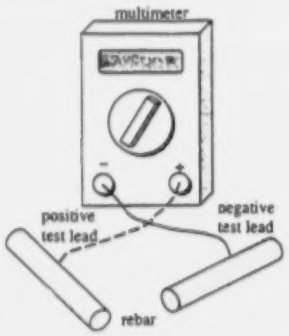
h. If all tests exhibit discontinuity, change the stationary ground and repeat the process. This eliminates the erroneous conclusion that all bars are discontinuous when only the stationary bar is discontinuous to all other ECR.

7. Conduct a half-cell potential survey along each ECR as follows:

   a. Contact the steel bar by pressing a test lead against the steel or installing a threaded screw. A common ground can be used for a set of bars that are electrically continuous. An independent ground needs to be established for each discontinuous bar (can use a drilled hole created for continuity).

   b. Prepare the copper-copper sulfate half cell in accordance with ASTM C-876.

Hook up for reverse resistance measurement.



c. Measure the half-cell potential along each ECR at 150mm intervals on sound concrete using a copper-copper sulfate half-cell. No measurements shall be made in delaminated areas. Record the location of the measurement and measured potential to the nearest mV.

d. Identify the location of the most negative and the most positive potential measured in the sawn area.

8. Collect ECR cores in sawn areas as follows:

   a. At locations exhibiting the most negative and the most positive potential (identified in 7.d) reconfirm the exact location of the ECR.

   b. Collect a 100mm diameter x 150mm deep core from each of these locations, centering the core on the ECR. If the target location contains an intersection of two ECR's, extract both ECR in the core.

9. Collect rapid chloride permeability cores as follows:

   a. Select three sawn areas randomly as per Phase 1 Section A.4.

   b. Locate steel to avoid extracting in the core.

   c. Collect a 100mm diameter x 150mm deep RCPT core.

10. Inspect and document any anomalies (such as cracking) in the interior of all core holes.

11. Repair all core holes and drilled holes as per standard MTO practice. Label and prepare all cores for shipment. The cores need to be prepared such that they are not broken during shipment and that any contained moisture is not lost.

FIELD EVALUATION OF BARRIER WALLS

A. Visual and Delamination Surveys

1. Perform a visual survey and document all anomalies.

2. Perform delamination survey on all three surfaces (wall facing traffic, wall away from traffic, and the upper horizontal surface) and document the quantity and location of the delaminations.

B. Selection of ECR in Each Span

1. Select 3 vertical ECR as follows:

    a. Select 3 vertical ECR evenly distributed throughout the length of barrier wall in each span. Locate their position in concrete using a rebar locator and mark their location over the entire height.

    b. Expose the upper end of each vertical bar with a 19mm diameter drilled hole.

2. Select 3 horizontal ECR as follows:

    a. Select 3 horizontal ECR evenly distributed along the elevation of the barrier wall. Locate their position in concrete using a rebar locator and mark their location over a length of 6m.

    b. Expose one end of each horizontal ECR with a 19mm diameter drilled hole.

    c. Continue along each bar and drill another 19mm diameter drilled hole at a horizontal distance of 3m.

    d. Check continuity between these two drilled holes. If continuity is not detected, another drilled hole at a horizontal distance of 3m along the same bar shall be made and continuity shall be checked between these last two drilled holes. This procedure shall be repeated until continuity is established between two consecutive drilled holes along the same bar

C. Clear Concrete Cover Measurement

1. Measure and document clear concrete cover over the reinforcing steel to the nearest mm. Concrete cover shall be measured from the highest point of the bar to the top of the concrete surface. If the bar

is not centered in the hole, measure cover from the highest point accessible in the hole to the top of the concrete surface.

D. Half-cell Potential Survey

1. Perform half-cell measurements along each of the vertical and horizontal bars select bar at 150mm intervals as follows:

    a. Half-cell potential measurements shall be performed along the entire elevation of the vertical bar and along a 3m section of the horizontal bar enclosed by the two excavations that contain steel exhibiting electrical continuity with each other.

    b. Each vertical and horizontal bar shall have an independent ground connection. A contact to the steel bar can be manually provided by pressing a test lead against the steel or by installing a threaded screw in the steel.

    c. Prepare the copper-copper half cell in accordance with ASTM C-876.

    d. Measure half-cell potential along each ECR at 150mm intervals on sound concrete using a copper-copper sulfate reference electrode. No measurements shall be made in delaminated areas. Record the location of the measurement and measured potential to the nearest mV.

    e. Identify the location of the three most negative and the three most positive potentials measured along the barrier wall.

E. Core Sampling

1. Collect 6 cores containing ECR from each barrier wall. Three cores shall contain ECR exhibiting the three most negative half-cell potentials while the other three cores shall contain ECR exhibiting the three most positive half-cell potentials.

2. No two cores are to be taken from the same ECR. If more than one core sampling location falls on the same

ECR, the core shall be collected from another ECR exhibiting similar potential.

3. In addition, three cores shall be taken along a vertical line at elevations of 230, 460, and 690mm from the deck surface. These cores shall not contain any ECR.

4. Measure cover depth in all core holes and note any anomalies within the core hole.

5. Inspect and document any anomalies (such as cracking) in the interior of all core holes.

6. Repair all core holes and drilled holes as per standard MTO practice. Label and prepare all cores for shipment. The cores need to be prepared such that they are not broken during shipment and that any contained moisture is not lost.

ABUTMENT SAMPLE COLLECTION

A. Visual and Delamination Surveys

1. Perform a visual survey and document all anomalies.

2. Perform a delamination survey and document the quantity and location of the delaminations.

B. Selection of ECR

1. Select 3 vertical ECR and 3 horizontal ECR as follows:

   a. Select 3 vertical ECR evenly distributed throughout the length of abutment. Locate their position in concrete using a rebar locator and mark their location over a 6 m length

   b. Select 3 horizontal ECR evenly distributed along the elevation of the abutment. Locate their position in concrete using a rebar locator and mark their location over a length of 6m.

   c. Expose the upper end of each vertical bar and the one end of the horizontal bar with a 19mm diameter drilled hole.

d. Continue along each vertical and horizontal bar and make another 19mm diameter drilled hole at a distance of 3m from the previous one.

e. Check continuity between these two drilled holes. If continuity is not detected, another drilled hole at a distance of 3m along the same bar shall be made and continuity shall be checked between these last two drilled holes. This procedure shall be repeated until continuity is established between two consecutive drilled holes.

C. Clear Concrete Cover Measurement.

1. Measure and document clear concrete cover over the reinforcing steel to the nearest mm. Concrete cover shall be measured from the highest point of the bar to the top of the concrete surface. If the bar is not centered in the hole, measure cover from the highest point accessible in the hole to the top of the concrete surface.

D. Half-cell Potential Survey

1. Perform half-cell measurements along 3m length of each bar at 150mm intervals as follows:

   a. Each bar shall have an independent ground connection. Contact to the steel bar can be manually provided by pressing a test lead against the steel or by installing a threaded screw in the steel.

   b. Prepare the copper-copper half cell in accordance with ASTM C-876.

   c. Measure half-cell potential along each ECR at 150mm intervals on sound concrete using a copper-copper sulfate half-cell. No measurements shall be made in delaminated areas. Record the location of the measurement and measured potential to the nearest mV.

E. Core Sampling

1. Collect 4 cores containing ECR from each abutment. Two cores shall contain ECR exhibiting the two most negative half-cell potentials while the other two cores

shall contain ECR exhibiting the two most positive half-cell potentials.

2. No two cores are to be taken from the same ECR. If more than one core sampling location falls on the same ECR, the core shall be collected from another ECR exhibiting similar potential.

3. In addition, three cores shall be taken along a vertical line at elevations of 300mm from the bottom, mid-height, and 300mm from the top edge of the abutment. These cores shall not contain any ECR.

4. Inspect and document any anomalies (such as cracking) in the interior of all core holes.

5. Repair all core holes and drilled holes as per standard MTO practice. Label and prepare all cores for shipment. The cores need to be prepared such that they are not broken during shipment and that any contained moisture is not lost.

LABORATORY EVALUATION

A. Storage and Preparation of ECR Cores

1. If the cores are not to be tested within 24 hours of receipt by the laboratory, they shall be stored in a freezer below -10 ° C.

2. When cores are removed from the freezer they shall be thawed for minimum of 24 hours or to room temperature, whichever is earlier.

3. Document the core identification number and date of initial testing on data sheets.

4. Measure the temperature of each core. If the temperature is less than 25°C, allow further thawing until a minimum temperature of 25°C is attained.

5. Remove all moisture from the surface of the cores with lint-free, salt-free absorbent tissues to insure that the surface is dry.

6. Document core information including diameter, height[IV] (h_t), cover over the topmost rebar (C)[V], and height of concrete below the bottom most rebar (h_b). All lengths to be measured to the nearest mm.

7. Visually observe the core to identify concrete damage such as cracking, honeycombing, and evidence of deterioration such as rust staining and delamination.

8. Label the core in multiple locations with its core identification number.

B. Extraction of Epoxy Coated Bars

1. Dry saw cut a 20mm deep groove in two diagonally opposing quadrants.[VI]

2. Carefully break the core into two pieces along the sawcuts using a chisel and hammer so as not to damage the ECR.

3. Carefully remove the epoxy coated bars from their traces. Insure that any liquid present on the bars is not disturbed.

C. Bar Selection and Preparation

1. A maximum of two bars shall be evaluated from each core.

2. Select the bars to be tested using the following criteria:

3. If more than two layers of bars are found in a core, only the top two layers shall be selected for evaluation.

4. If a layer contains more than one bar, one bar will be randomly selected for evaluation.

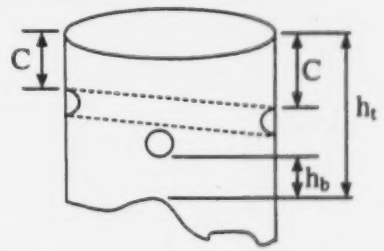
5. Any bars that are not completely extracted (entire cross section) shall not be tested.

6. With a black permanent marker, paint the top half of one end of each selected bar to designate its orientation in the core.

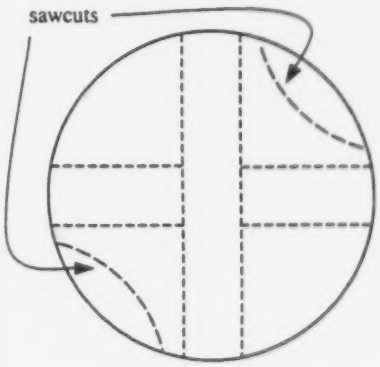
7. Drill and tap the other end of each selected bar (the end that is not marked with a black marker). Install a screw and attach a pre-labeled tag.[VII]

[IV] The height of a core may vary from one location to another around the perimeter of the core if both the upper and lower surfaces are not smooth. The maximum core height should be recorded as the maximum height that contains the entire cross section of the core.

[V] The concrete cover depth is to be measured from both ends of the topmost bar.



[VI] Sawcut for extraction of bars



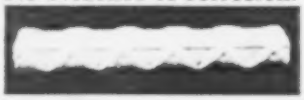
[VII] When there are more than one bar extracted from the core the top bar in the mat and the bottom bar should be identified as such.

D. **Visual Inspection of Epoxy Coated Bars**

1. Dry the epoxy coated rebar if necessary by wiping with lint-free, salt-free absorbent tissues.

2. Record bar size, length, deformation pattern, and coating color for each ECR..

3. Visually observe the condition of the coating and the reinforcing steel and rate as follows:

Rating	Description
1	No evidence of corrosion. 
2	A number of small, countable corrosion spots.
3	Corrosion area less than 20% of total ECR surface area.
4	Corrosion area between 20% to 60% of total ECR surface area.
5	Corrosion area greater than 60% of total ECR surface area.

4. If the rating of the bar is equal to or greater than 4 no further testing needs to be performed.

5. Record the number of mashed areas, bare areas, and blisters for each rebar. Include comments for each rebar as necessary.

E. **Holiday Detection**

1. Attach the ground wire of the holiday detector to the screw that was previously installed on the rebar.

2. KTA-Tator M/1, 60V holiday detector with a ¼ inch square sponge shall be used.

3. Perform holiday detection test on all surfaces of each bar in accordance with ASTM 62 - Method A.

4. When recording the number of holidays, distinguish between holidays located in bar ID areas from those not located in bar ID areas.[viii]

[viii] During holiday detection, bare areas will also produce a beep. Areas previously categorized as bare areas will not be counted as holidays. If a holiday is detected over a mashed area the mashed area will be re-categorized as a bare area.

If a continuous beep is detected along a bar, then the number of holidays will be considered to be equal to the length of bar along the test trace divided by the width of sponge.

F. Coating Adhesion

1. Measure coating adhesion in accordance with NACE TM0185 - Section 5.3.2, "Knife Adhesion Testing," with four modifications.

   a. The sample will not be heated in an autoclave.

   b. Use an "X" shaped groove in place of the "V" groove described.

   c. Use a modified rating scale.

2. Conduct wet adhesion test in three undamaged areas[ix].

3. If damaged coating areas are identified on the bar, conduct up to three additional wet adhesion tests adjacent to the damaged areas.

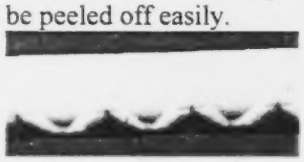
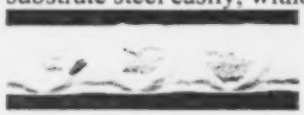
4. Record visual observations of the steel beneath the coating.

5. Rate all adhesion test areas as follows:

[ix] The knife adhesion test conducted without drying in a desicator is termed the wet adhesion test. This test measures coating adhesion in an as-is condition.

The wet adhesion test must be completed within 24 hours from the time the rebars are extracted from the core.

Rating	Description
1	Well adhered coating that cannot be peeled or lifted from the substrate steel.

3	Coating that can be pried from the substrate steel in small pieces, but cannot be peeled off easily. 
5	Coating that can be peeled from the substrate steel easily, without residue. 

Note: Sections B through F must be completed within a 24 hour period.

G. Preparation of Powdered Concrete Samples for Chloride Analysis

1. For cores with ECR prepare concrete powder samples as follows:

   a. Cut the portion of the core above the epoxy coated rebar, along its long axis, into two pieces of approximately half the diameter. Cut one piece into 10mm thick semi-disks and powder for chloride analysis. Measure down 5mm from the top of the other piece and repeat the process of cutting 10mm thick semi-disks[X].

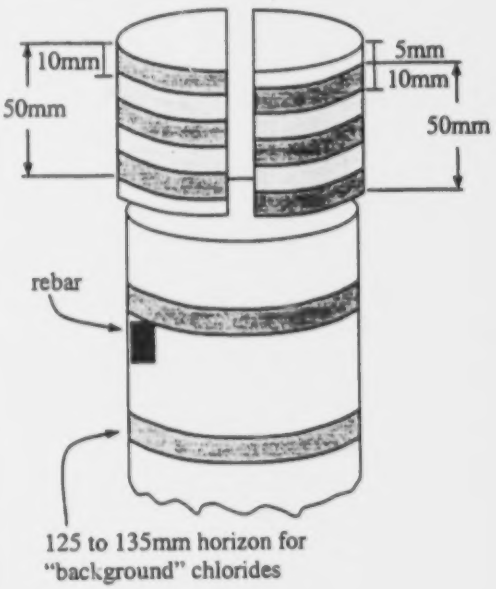
   b. Cut a 10mm disk of the concrete immediately above the top rebar and powder the sample.

   c. Cut a 10mm thick disk located between 125 to 135mm depth of the core and powder the sample.

2. Prepare powdered concrete samples from cores without ECR as follows:

   a. Cut the core, along its long axis, into two pieces of approximately half the diameter. Cut 10mm disks from one half starting at the top of the concrete core. On the second half, cut 10mm disks offset from the top of the core by 5mm.

[X] Preparation of Powdered Concrete Samples



H. Chloride Analysis

1. Perform total chloride ion analysis of all powdered samples in accordance with MTO procedures.

I. Rapid Chloride Permeability Test

1. Use a water-cooled saw to cut a 50mm slice representing the 20-70mm horizon below the surface. The cut should be made parallel to the top surface of the core.

2. Perform the rapid chloride permeability test in accordance with AASHTO T 277-93.

3. In addition to the procedures listed in the Standard Test Method, measure AC resistance prior to the application of 60V and 1 minute after the application of 60V. From the average of these measurements calculate resistivity in ohm-cm using a form constant.[x]

4. If the thickness of the sample is not exactly 51mm apply the following correction factor to each measured current:

$$I = I_m L_a / 51$$

where I_m is the measured current in mA, L_a is the actual sample thickness in mm and I is the corrected current in mA.

[x] The form constant is calculated as

$$F = L/A$$

where F is the form constant, A is the surface area of the core in cm^2 and L is the height of the core in cm.

APPENDIX B
Problems Associated with Determining the Corrosion Initiation Period

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The time to initiation, t_i, is defined as the period between initial construction and the onset of corrosion of the embedded steel reinforcement. For a reinforced concrete highway structure (e.g. bridge deck, parapet walls, substructure elements) exposed to chloride-containing de-icing salts, t_i represents the time taken for chlorides applied to the external surface to penetrate through the concrete cover and accumulate at the embedded steel in sufficient quantity to breakdown the passive layer and thereby activate the corrosion process. The length of time required for this process is dependent of a number of parameters including the following:

- Exposure conditions at the surface of the reinforced concrete element (e.g. frequency and quantity of salt applications, temperature, humidity, precipitation). In any given structure, the exposure conditions can vary widely.
- Presence and efficiency of surface barriers (e.g. membranes)
- Concrete quality (in terms of its ability to resist the penetration of chloride ions)
- Depth of clear concrete cover to the embedded steel reinforcement
- Nature and extent of cracking in the concrete cover
- Amount of chlorides required to initiate corrosion, C_t

The level of chlorides required to initiation corrosion is often referred to as the chloride threshold, C_t. There is no consensus regarding the actual value of the C_t and values ranging from 0.02 to 0.30% (mass % of concrete) have been reported in the literature (adapted from Glass & Buenfeld, 1995)[3]. The actual relationship between corrosion and chloride content is likely to be influenced by a whole range of parameters including the type, composition and quantity of Portland cement and other supplementary cementing materials, the moisture content and temperature inside the concrete, the porosity and pore structure of the concrete, the nature (composition) of the steel surface, and the presence of other species in the pore solution (e.g. alkalis or chemical inhibitors). At this time there are no clearly defined relationships that can easily be incorporated into a simple service life model and it is common to use a single value. For example, the Ontario Ministry of Transportation (MTO) typically use a value of C_t = 0.025%, which can be regarded as a very conservative limit. The same value will be used in the subsequent analyses.

In most mathematical models for predicting the service life of reinforced concrete exposed to chlorides the effect of concrete quality is included by assigning an "apparent" diffusion coefficient to the concrete material and assuming that ionic diffusion is the sole mechanism of chloride transport. This is, of course, an oversimplification, but in the absence of a more comprehensive knowledge of the role of other transport mechanisms, the approach is reasonable for many types of structures and exposure conditions.

One of the earliest mathematical models (as opposed to empirical models) was that of Browne[2] (1980) who proposed the use of a standard closed-form solution of Fick's second law for determining diffusion coefficients in existing concrete structures exposed to chlorides and for

predicting the time to corrosion for steel at various cover depths. The standard solution may be written as follows:

$$\frac{C_x}{C_s} = 1 - erf\left(\frac{x}{2\sqrt{D_a \cdot t}}\right) \qquad \text{Eqn. 1}$$

where: C_s = chloride concentration at the surface
C_x = chloride concentration at distance x and time t
x = depth from the surface
D_a = 'apparent' diffusion coefficient

This approach is still commonly used today. If the diffusion coefficient, D_a, is known and the surface concentration, C_s, can be estimated (e.g. from existing field studies), Eqn. 2 can be used to predict the chloride concentration profile at any time, t. An example of this is shown as Fig.1 using values of C_s = 0.60% (mass % of concrete) and $D_a = 2.0 \times 10^{-12}$ m^2/s.

Eqn. 1 can be used to predict the residual time to corrosion in existing structures. This is usually done by establishing a chloride concentration profile on cores taken from the structure and curve-fitting Eqn. 1 to the experimental data. This yields the best-fit values for the two unknowns C_s and D_a. These parameters are then used as inputs to Eqn. 1 to establish the time, t_i, at which the chloride threshold, C_t, reaches the depth of the steel, x_d.

Such an approach can be simplified still further. On examination of Eqn. 1 it can be seen that for the condition where C_s and D_a are constant with time (which is an explicit assumption for the numerical solution) the rate of penetration of a specific chloride concentration value (e.g. the threshold, C_t) obeys a root-time relationship. This is shown in Fig. 2, where the depth of penetration of the chloride threshold, C_t = 0.025%, is plotted against the square root of time. The data points shown are actually the values where the horizontal line C = 0.025% intersects the profiles at t = 1, 5, 10, 25 and 50 years on Fig. 1. It is clear that the depth of penetration of C_t is directly proportional to the age of the structure. One can use this simple relationship to determine the time to corrosion of an existing structure. First the concentration profile at the present time, t_p, is determined and from this profile the depth of penetration, x_p, of the chloride threshold, C_t, is estimated. The predicted age at corrosion, t_i, is the time when the same concentration, C_t, reaches the depth of the steel, x_d, and is calculated using:

$$t_i = t_p \cdot \left(\frac{x_d}{x_p}\right)^2 \qquad \text{Eqn. 2}$$

There is no need to use Eqn. 1 and the two approaches will yield the same result (assuming reasonable concentration profiles are obtained).

Although Eqns. 1 and 2 are oversimplified, they represent the only reasonable approach to determining the time to corrosion in a structure where nothing else is known about the concrete or its exposure condition (i.e. only the chloride profile at the present time is known).

In reality, Eqn. 1 is not strictly applicable to concrete highway structures exposed to periodic application of de-icing salts for a number of reasons, chief among which are that neither the diffusion coefficient, D_a, nor the surface concentration, C_s, are constant with time.

The concentration of chlorides in the surface layers of concrete exposed to periodic applications of salt will increase with time as more and more salt is applied. It has been shown for bridge decks that the time-dependant relationship can be represented by various mathematical functions such as that shown by the solid line in Fig. 3. (Actually, there are also likely to be seasonal variations, which further complicate the relationship). Such a relationship can be simply represented by a bilinear relationship, whereby the surface concentration increases linearly with time until it reaches a maximum value, C_{max}, and remains constant with time thereafter; this is shown by the broken line in Fig. 3. Weyers[4] has shown this to be a reasonable approximation for bridge decks in Virginia and such a relationship is used in the current version of the service-life model, Life-365.

The diffusion coefficient of concrete, like other physical properties, is not constant with time, but decreases as the concrete matures. Recent studies (Bamforth, 1998[5]; Thomas and Bamforth, 1998[6]) have demonstrated that the extent of the reduction in the diffusion coefficient with time can be substantial, especially in concretes containing slowly reacting pozzolans such as fly ash or latent hydraulic materials such as ground granulated blastfurnace slag. Such concretes may actually have relatively high diffusion coefficients at early ages and, if the time-dependent decay in diffusivity is not accounted for, predictions based solely on early-age measurements may be expected to be in error.

A number of workers have shown that the relationship between diffusivity and time is best described by a power law (Bamforth, 1998[5]; Thomas and Bamforth, 1998[6]; Tang and Nilsson 1992[7]; Mangat and Molloy, 1994[8]; Maage et al. 1995[9]), where the exponent is potentially a function of both the materials (e.g. mix proportions) and the environment (e.g. temperature and humidity). The following equation, which is incorporated in the model used for this study, is an example of such a relationship:

$$D_t = D_{ref} \cdot \left(\frac{t_{ref}}{t} \right)^m \qquad \text{Eqn. 3}$$

where: D_t = diffusion coefficient at time t
D_{ref} = diffusion coefficient at reference time, t_{ref} (typically 28 days)
m = constant

A typical relationship with the reference diffusion coefficient at t = 28 days being represented by $D_{28} = 1.0 \times 10^{-11}$ m^2/s and the decay coefficient by $m = 0.30$ is shown in Fig. 4.

There is no simple mathematical equation for predicting the chloride concentration profiles at various ages for concrete in which both the values of C_s and D_a vary with time. However profiles can be predicted using a finite difference solution. Fig. 4 shows profiles at different ages predicted using a mathematical model, Conflux, developed at the University of Toronto (Boddy

et al. 1998) using the time-dependent relationships for C_s and D_a shown in Figs. 3 (broken line) and 4, respectively.

The model, Conflux, can also be used to predict the depth of penetration of the chloride threshold, C_t, at various times. An example of this, for the conditions described above, is shown by the solid line in Fig. 5 using the square root of time as the abscissa. It can be seen that the depth of penetration is no longer a linear function of root time. It can be shown that the deviation from a straight line is a function of the extent to which C_s and D_a vary with time. Fortunately, the time dependent nature of C_s and D_a tend to compensate for one another, as the surface concentration is low when the diffusion coefficient is high and vice-versa. As a result, the deviation from a straight-line relationship is not too large for typical cases. If the depth of the chloride threshold is determined in an existing structure then extrapolating the result to predict the initiation period using Eqn. 2 should not result in too large an error, provided the present age at the structure (i.e. the time at which the chloride profile is established) is sufficiently old. For example, the broken line in Fig. 5 shows the error in fitting a root-time relationship to the data determined at 10 years of age. The error between the broken straight line and the solid curved line beyond 10 years is almost certainly acceptable for most prediction purposes. The deviation between these two lines becomes greater as the age of the structure at the time of sampling decreases.

The discussion so far has focussed on unprotected reinforced concrete structures. When a protective membrane is applied to the surface of the structure predictive modelling is much more complex because of the uncertain performance of the membrane in terms of screening chlorides from the surface of the concrete. Fig. 7 depicts how the chloride level in the surface of the concrete might change with time if a protective membrane is applied at the time of construction. The membrane provides a partial screen to chlorides for a limited period of time; in the example shown this is assumed to be 15 years for the purpose of illustration. During this 15-year period only a limit quantity of chlorides are permitted to pass through the membrane. Beyond this time, the surface chloride content begins to increase as the condition of the membrane deteriorates and inevitably the surface chloride content will attain similar values as an unprotected concrete unless the membrane is either maintained or replaced. Unfortunately, there are insufficient data available to permit the time-dependant nature of the surface chloride concentration to be predicted with any degree of confidence. Neither the life span of the membrane (assumed to be approximately 15 years in Fig. 7) nor the rate of deterioration of the membrane and consequent rate of increase in the surface chloride content can be predicted. In other words, it is not possible to construct a relationship similar to that shown in Fig. 7 except in a purely hypothetical case.

Despite the above-mentioned shortcomings, the effect of a membrane with the hypothetical performance shown in Fig. 7 is illustrated here using the model Conflux. Fig. 8 shows how, with such a protective membrane, the chloride profile will advance with time for a concrete with a time-dependent chloride diffusion coefficient matching the relationship shown in Fig. 4. During the first ten to fifteen years very little chloride penetrates the concrete, but as the membrane begins to break down, the profile advances relatively rapidly. The solid line in Fig. 9 shows the relationship between the depth of penetration of the chloride threshold and the square root of time. The chloride threshold is not attained at any location within the concrete for a certain period of time (approximately 10 years for the example shown). Beyond this period the threshold value penetrates the concrete steadily although the rate of penetration is not linear with root time.

However, as with the former case, the deviation from a linear relationship (once the threshold exhibits significant penetration) is not too severe for the example shown. A number of analyses have been performed, which indicate that assuming a linear relationship beyond the point of degradation of the membrane is a reasonable approximation provided the rate of deterioration of the membrane is not too slow or the rate of decrease of the concrete's diffusion coefficient is not too high. However, in the case of a membrane-protected system, it is no longer appropriate to assume a linear relationship that passes through the origin. Such a relationship is shown in Fig. 9 (broken line) for the case where the depth of penetration of the threshold is determined for the case of a 20-year-old structure. The assumed relationship does not provide a reasonable approximation of the future rate of penetration.

The above discussion serves as an illustration of the problem for a deck with a protective membrane. Without detailed information regarding the performance of the membrane in terms of its lifespan and efficiency in screening chlorides it is not possible to utilise existing mathematical models to predict the performance of the system.

However, if the structure is sampled at an age beyond which significant chloride penetration has occurred, it may be possible to determine the rate of chloride ingress (as defined as the rate of penetration of the threshold concentration, C_t) using additional data. For example, consider the case shown in Fig. 9 when a concrete element is sampled at 20 years of age and the depth of penetration of C_t is determined to be 25 mm (marker shown in Fig. 9). In the case of an unprotected deck the quality of the concrete is essentially a function of the amount of penetration during the period of exposure and one can predict the time to corrosion with an acceptable degree of certainty using Eqn. 2. In the case of a deck protected by a membrane, the amount of penetration is a function of both the membrane efficiency and the concrete quality, and it is not possible to deconvolute the two effects. However, if in addition to establishing the depth of penetration of the threshold at the time of sampling, some measure could be made to determine the slope of the line that passes through this point and describes the rate of penetration once the membrane has failed, then it may be possible to determine the time to corrosion within a reasonable degree of confidence. This concept is shown in Fig. 10 for the case when the depth of threshold penetration is determined on a 20-year-old structure. The broken line shows the predicted rate of penetration into the concrete once the membrane has "failed". If the membrane is sufficiently dysfunctional, then the rate of chloride penetration will largely be a function of the concrete quality at that time and the slope of the line may be determined from an appropriate test conducted on a concrete sample taken from the structure. Such a test could be a laboratory diffusion test or a simple rapid indicator of diffusion.

Proposed Approach for Determining the Corrosion Initiation Period

Fig. 11 shows the rate of penetration of the chloride threshold using the surface concentration build-up shown in Fig. 7 for three different concretes. These concretes were chosen to be represent three different concretes with a range of diffusion properties (D_{28} and m values) as follows:

	D_{28} (m^2/s)	m	Typical Mix
Case 1	8.0×10^{-12}	0.20	Portland cement; w/cm = 0.40
Case 2	8.0×10^{-12}	0.40	25% Fly Ash; w/cm = 0.40
Case 3	2.4×10^{-12}	0.34	6% Silica Fume + 25% Slag, w/cm = 0.36

Also shown in Fig. 11 are markers indicating the theoretical depth of penetration of C_t assuming the concretes were sampled at t_p = 25 years and broken lines representing the best-fit linear relationships describing the penetration of C_t as a function of the square root of time. This yields the following data:

	x_p at t_p = 25 years	Slope of depth of penetration vs. root time, k ($mm/y^{0.5}$)	Theoretical D_p values at t_p = 25 years (m^2/s)
Case 1	53.0 mm	40.0	2.5×10^{-12}
Case 2	30.0 mm	17.5	7.9×10^{-13}
Case 3	19.5 mm	11.5	4.2×10^{-13}

In the approach proposed the value of k is determined from the diffusion coefficient (or alternative accelerated test) measured on cores taken from the structure at t_p = 25 years. The theoretical values of the 25-year-old diffusion coefficients, D_p, are also given in the table above.

The relationship between the measured diffusion coefficients and the rate of penetration, k, is shown in Fig. 12; this relationship is represented by the following mathematical equation:

$$k = 5 \times 10^9 \cdot D_p^{0.7} \qquad \text{Eqn. 4}$$

An alternative approach to measuring the diffusion coefficient is to use the result from the ASTM C 1202 or rapid chloride permeability test (RCPT) measured on cores taken from the structure. This is probably a preferred approach considering the capabilities of testing laboratories in Ontario (only two or three labs can perform diffusion tests whereas many can carry out the rapid test. Despite criticism of the rapid test, there does exist a broad correlation between the RCPT and non-steady-state diffusion tests. Fig. 13 shows the relationship for 56 different concrete

samples representing mixes with a wide range of mix proportions and various ages. The relationship between the RCPT value and D can be expressed as follows:

$$D = 2\times10^{-15} \cdot RCPT^{1.1} \qquad \text{Eqn. 5}$$

The relationship between the measured RCPT value and the rate of penetration, k, is approximately linear as shown in Fig. 14; this relationship is represented by the following mathematical equation:

$$k = 0.05 \cdot RCPT \qquad \text{Eqn. 6}$$

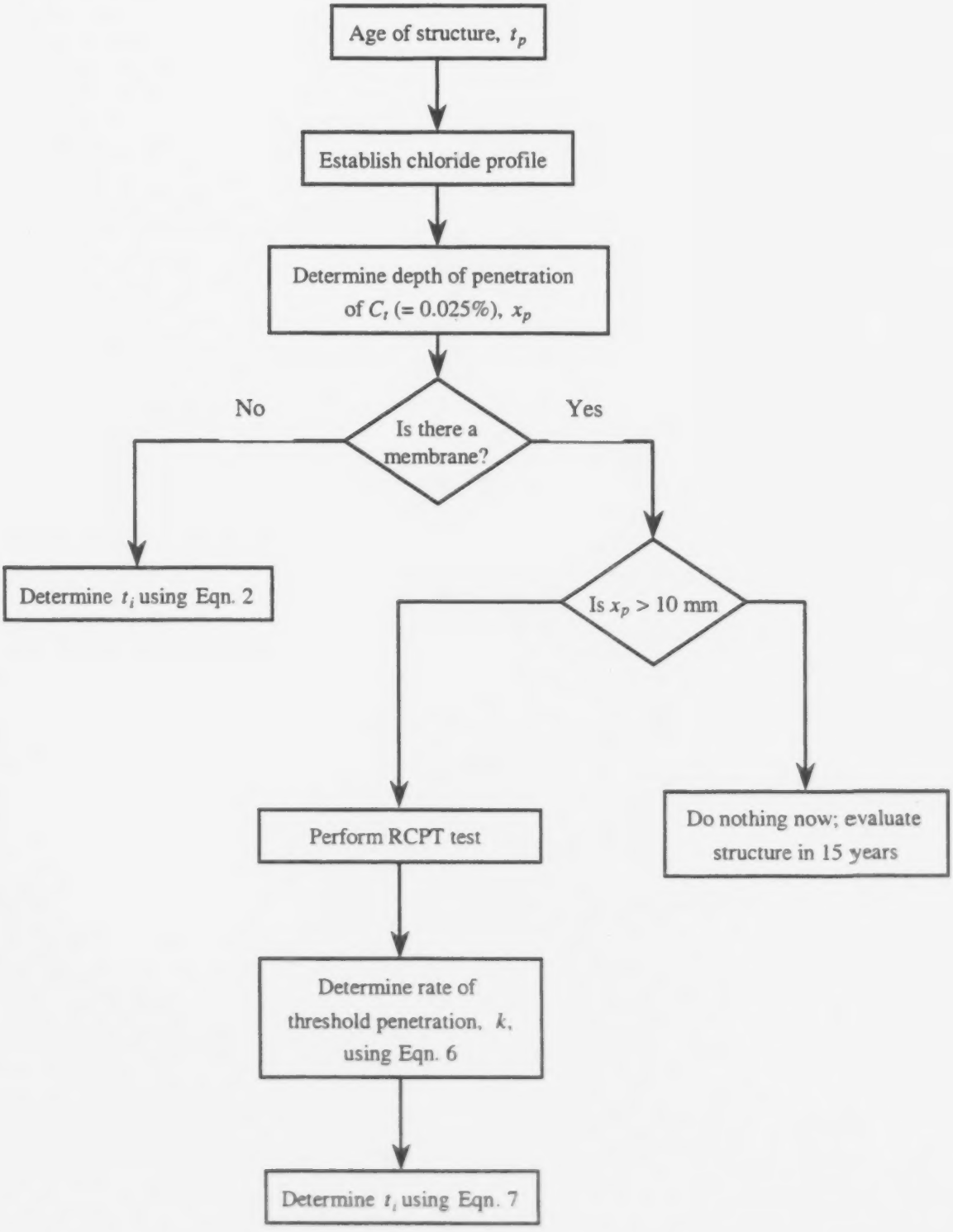
The time to corrosion initiation can be calculated from the chloride profile and RCPT value determined on cores taken from a structure by using the following formula:

$$t_i = \left(\frac{x_d - x_p}{k} + \sqrt{t_p} \right)^2 \qquad \text{Eqn. 7}$$

where: t_i = age of structure when corrosion initiates (years)
t_p = age of structure at time of sampling (years)
x_d = depth of cover (mm)
x_p = depth of threshold, C_t, at t_p (mm) determined from chloride profile
k = rate of threshold penetration (mm/y$^{0.5}$) determined from RCPT (Eqn. 6)

This approach is only suitable for elements with protective membrane systems and when the threshold chloride value ($C_t = 0.025\%$) has penetrated a significant distance into the concrete (i.e. > 10 mm). For unprotected elements the use of Eqn. 2 is recommended. In cases where the threshold value has penetrated less than 10 mm in elements with a membrane system, it is likely that the residual time to corrosion is greater than 15 years and, as such, a further evaluation is recommended in 15 years time.

Flow Chart for Determining the Corrosion Initiation Period


Age of structure, t_p
Establish chloride profile
Determine depth of penetration of C_t (= 0.025%), x_p
Is there a membrane?
No
Yes
Determine t_i using Eqn. 2
Is x_p > 10 mm
Perform RCPT test
Do nothing now; evaluate structure in 15 years
Determine rate of threshold penetration, k, using Eqn. 6
Determine t_i using Eqn. 7

References

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(4) Weyers, R. (a). ***Ref to be supplied***.

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(6) Thomas, M.D.A. and Bamforth, P.B. "Modelling chloride diffusion in concrete; effect of fly ash and slag." Cement and Concrete Research, Vol. 29, pp. 487-495.

(7) Tang, L. and Nilsson, L-O. "Chloride diffusivity in high strength concrete at different ages." Nordic Concrete Research, 1992, pp. 162-171.

(8) Mangat, P.S. and Molloy, B.T. "Prediction of long term chloride concentrations in concrete." Materials and Structures, Vol. 27, 1994, pp. 338-346.

(9) Maage, M., Helland, S. and Carlsen, J.E. "Practical non-steady state chloride transport as a part of a model for predicting the initiation period." Chloride Penetration into Concrete, (Ed. L.-O. Nilsson and J. Ollivier), pp. 398-406.

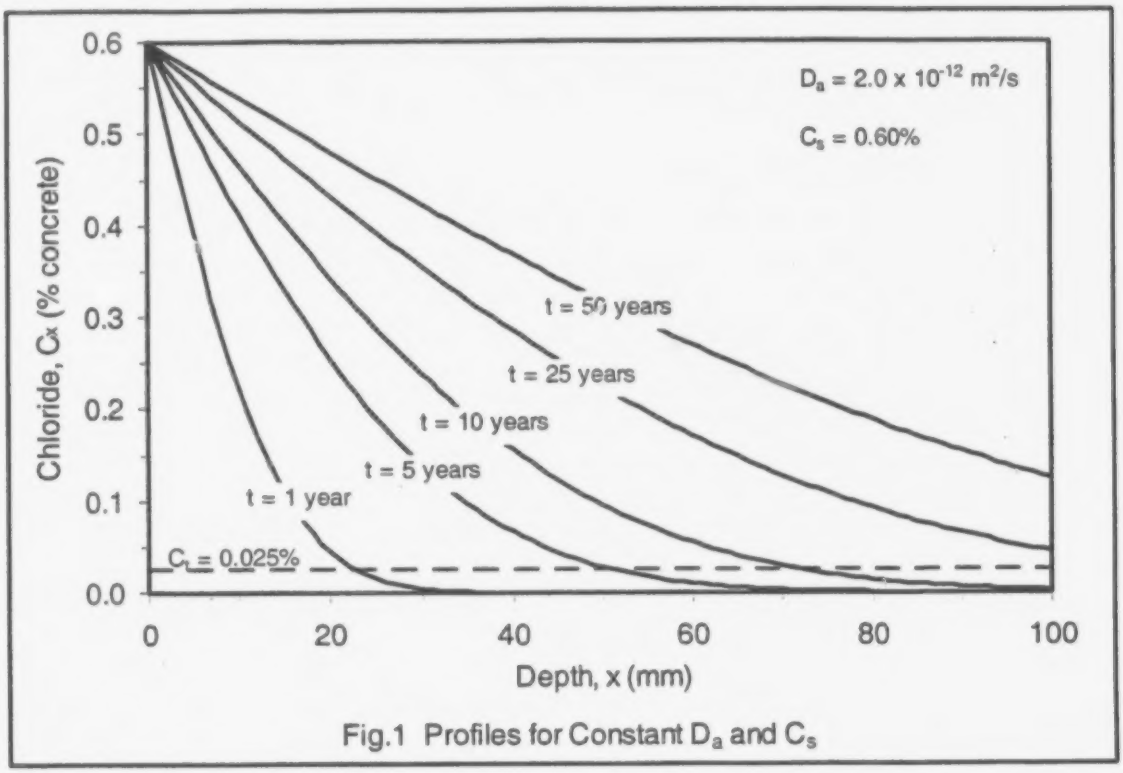


Fig.1 Profiles for Constant D_a and C_s

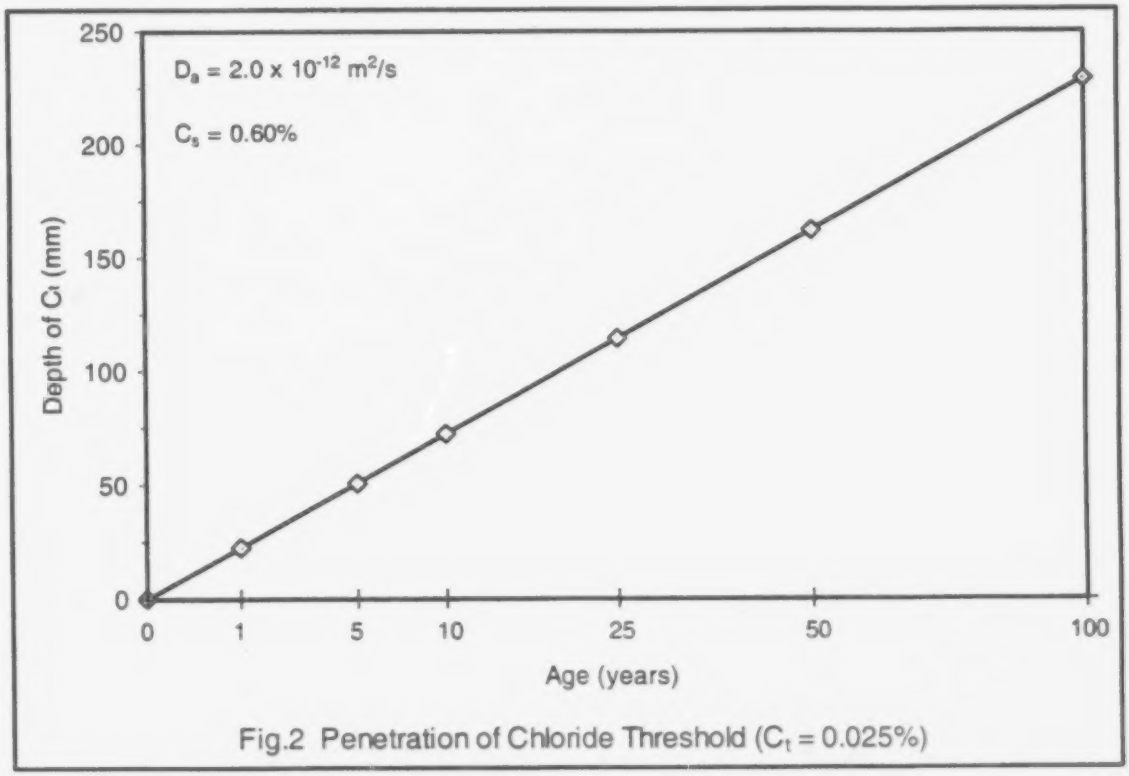


Fig.2 Penetration of Chloride Threshold (C_t = 0.025%)

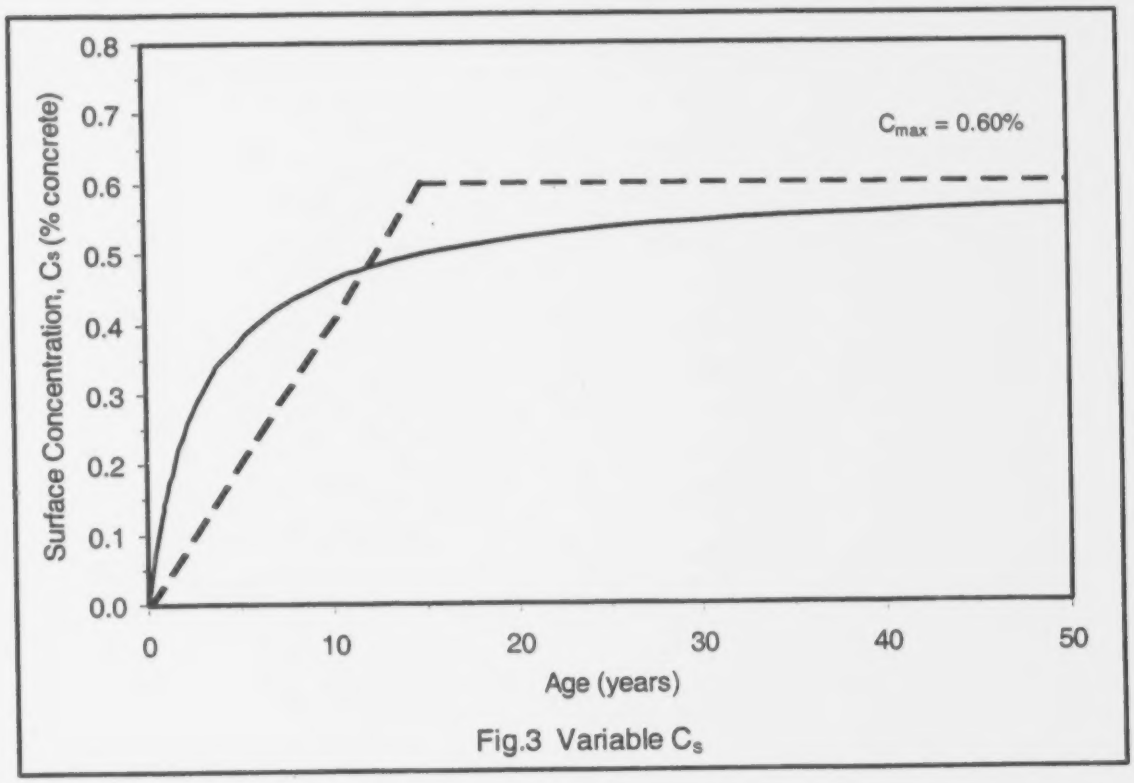


Fig.3 Variable C_s

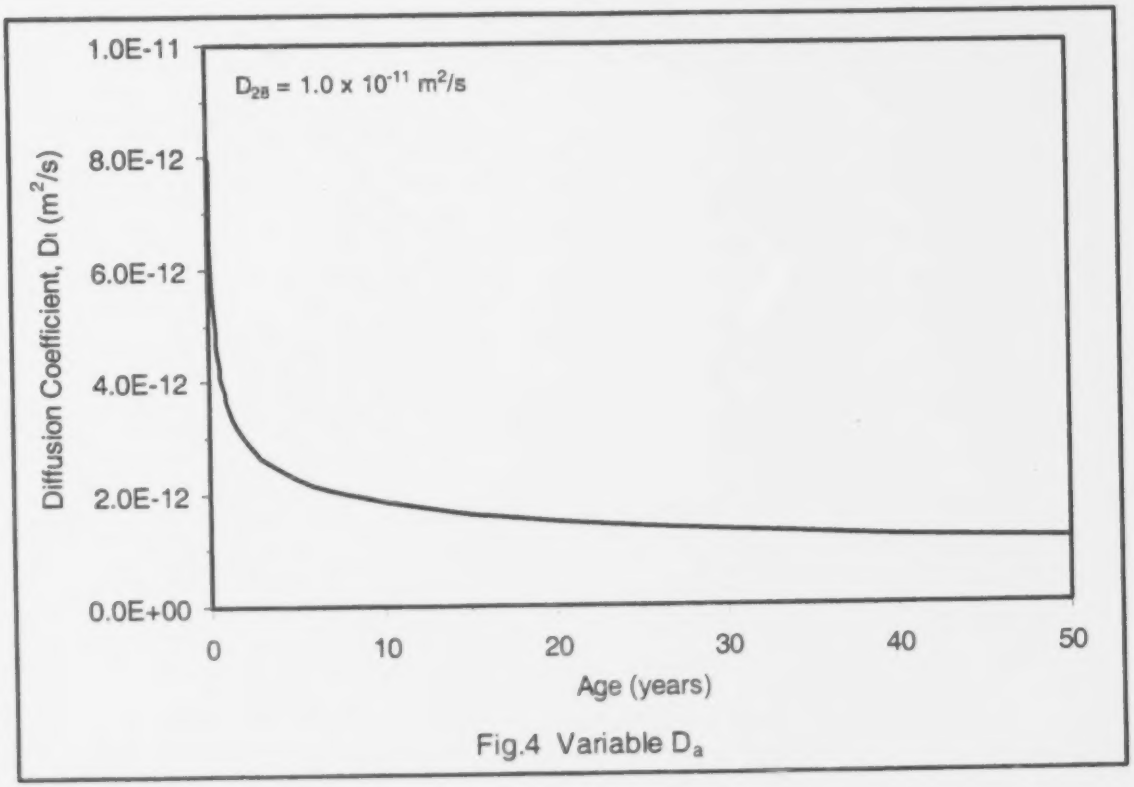


Fig.4 Variable D_a

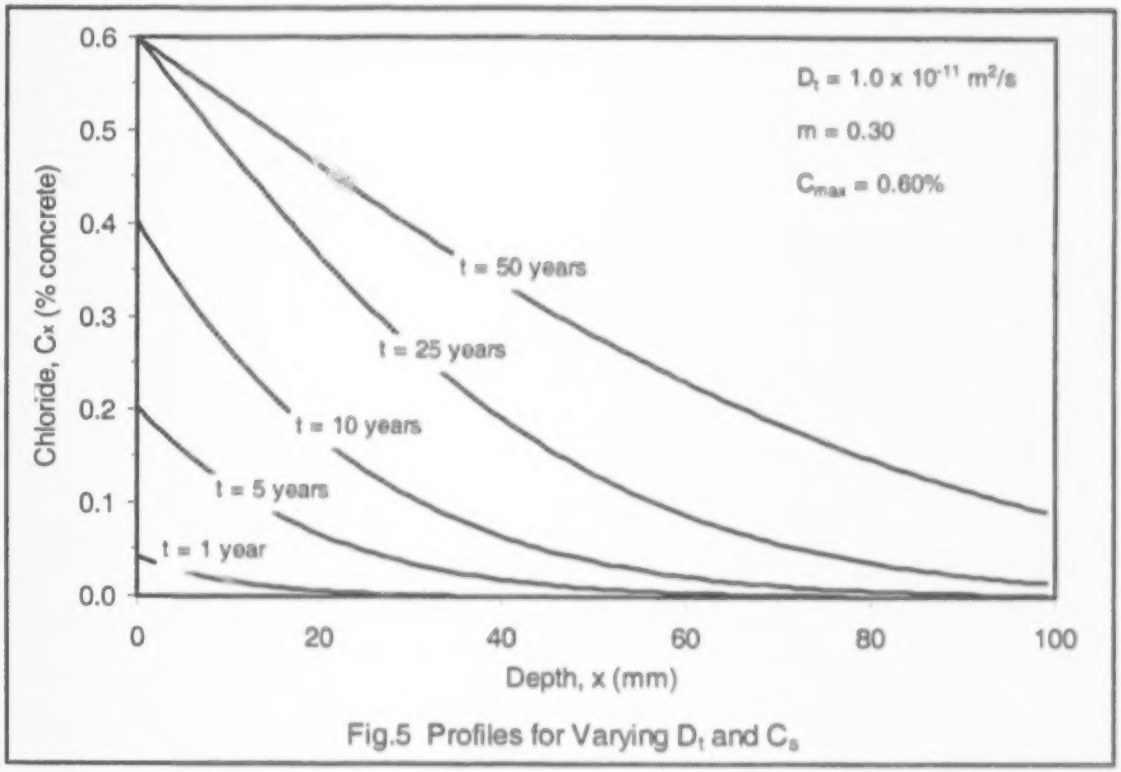


Fig.5 Profiles for Varying D_t and C_s

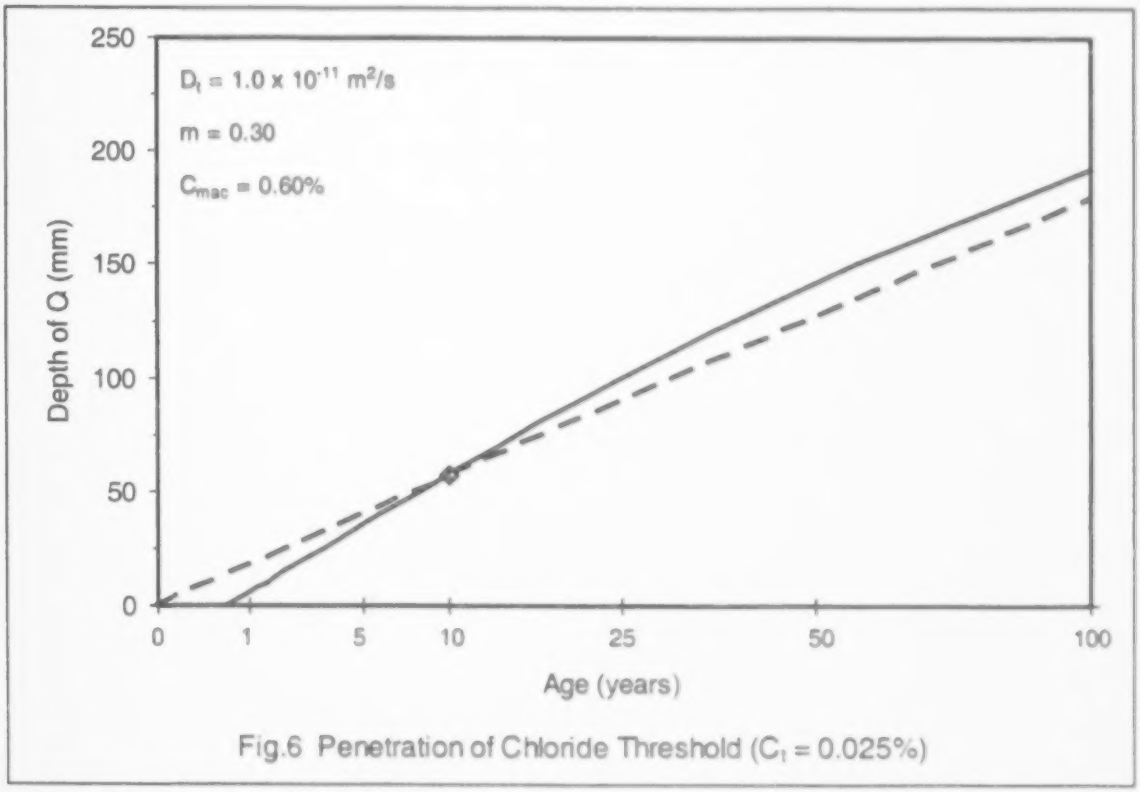


Fig.6 Penetration of Chloride Threshold (C_t = 0.025%)

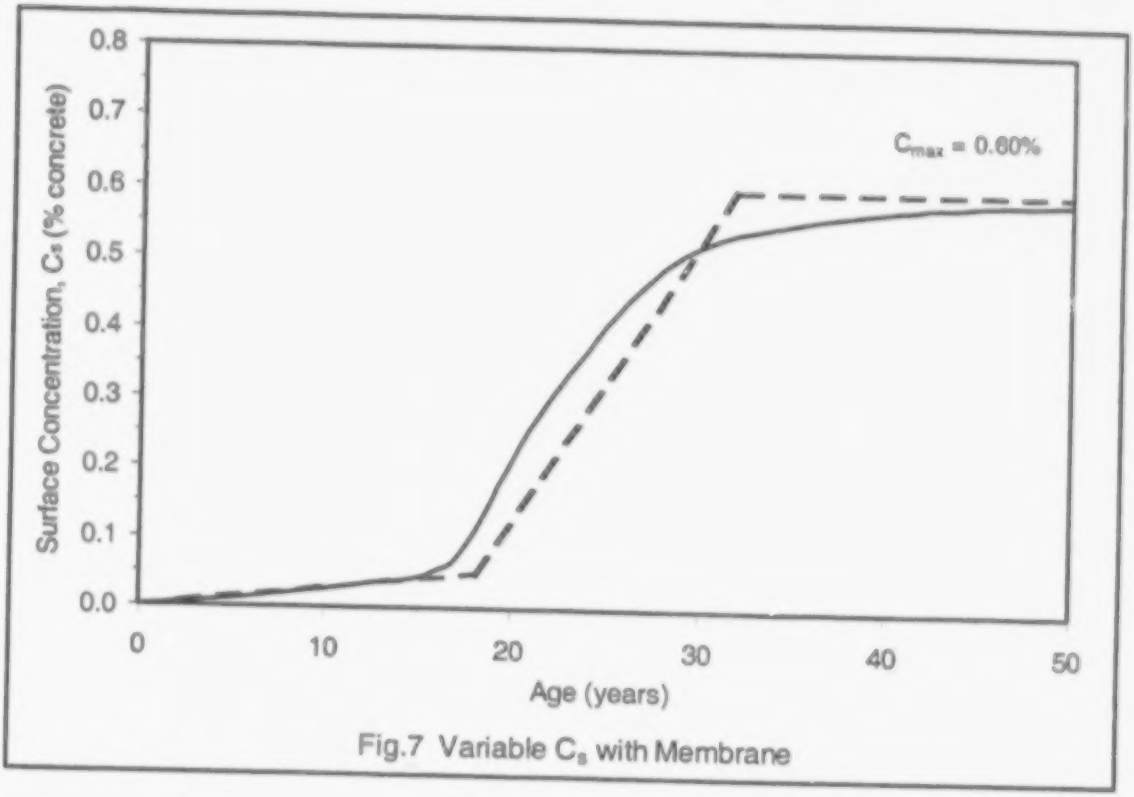


Fig.7 Variable C_s with Membrane

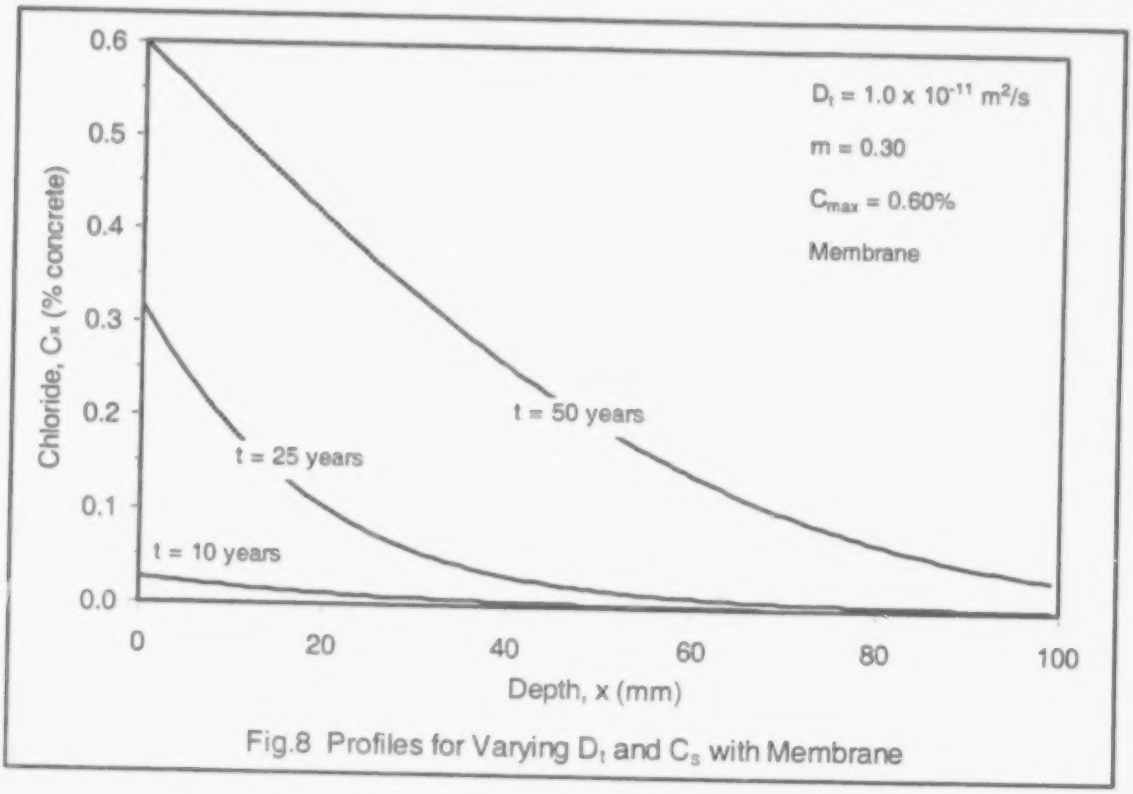


Fig.8 Profiles for Varying D_t and C_s with Membrane

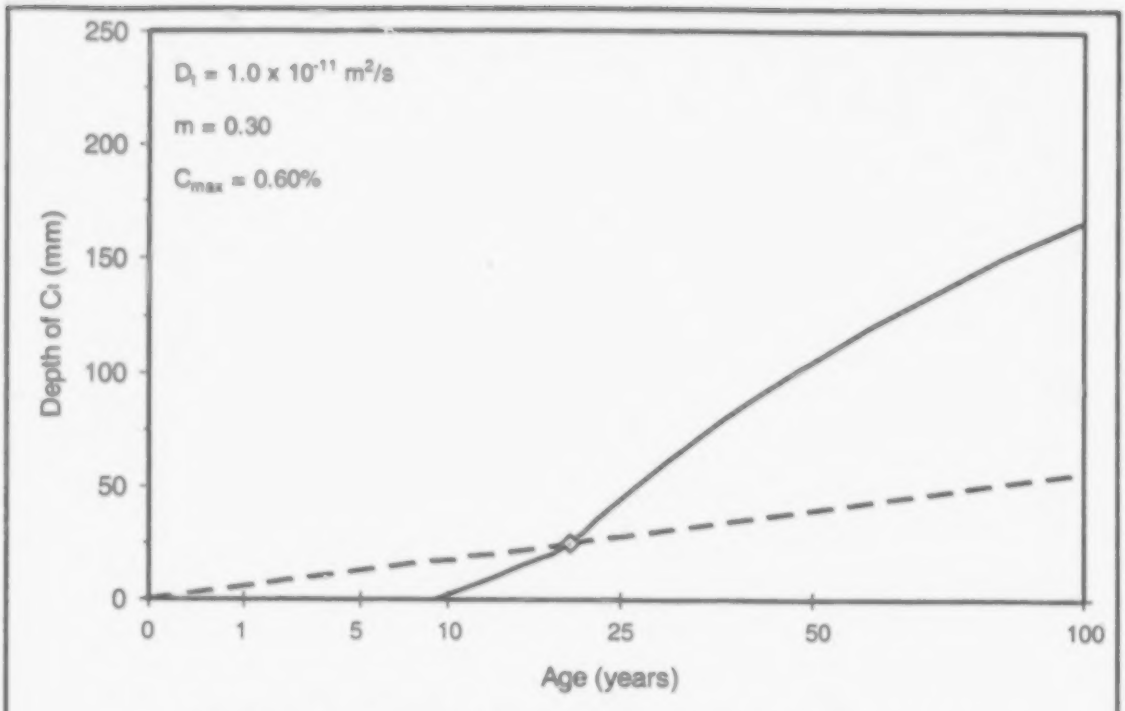


Fig.9 Penetration of Threshold (C_t = 0.025%) with Membrane

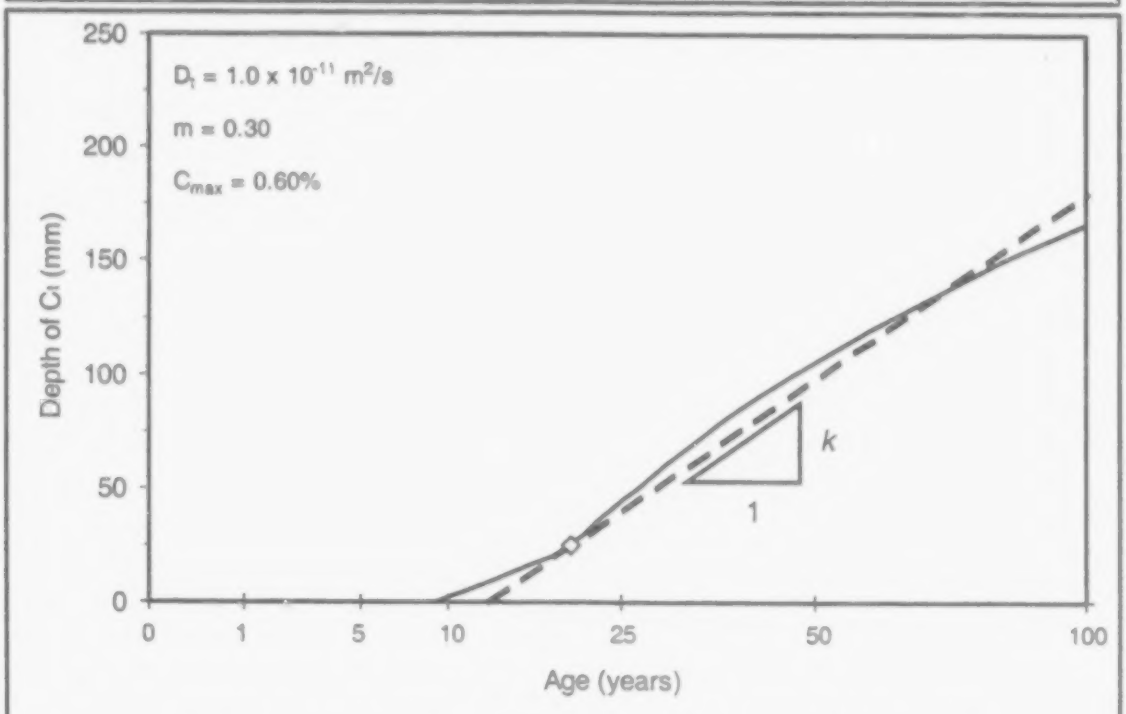


Fig.10 Penetration of Threshold (C_t = 0.025%) with Membrane

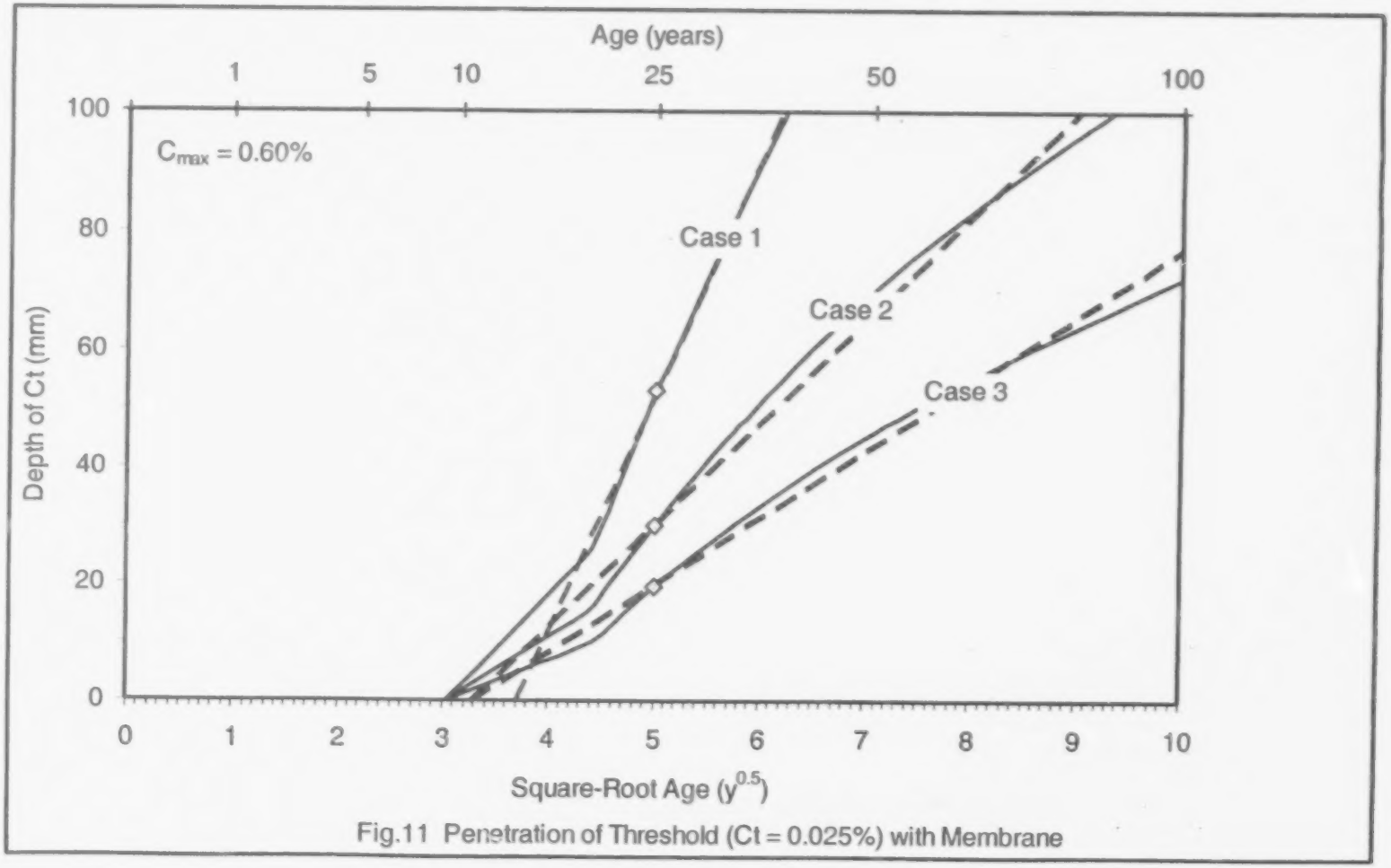


Fig.11 Penetration of Threshold (Ct = 0.025%) with Membrane

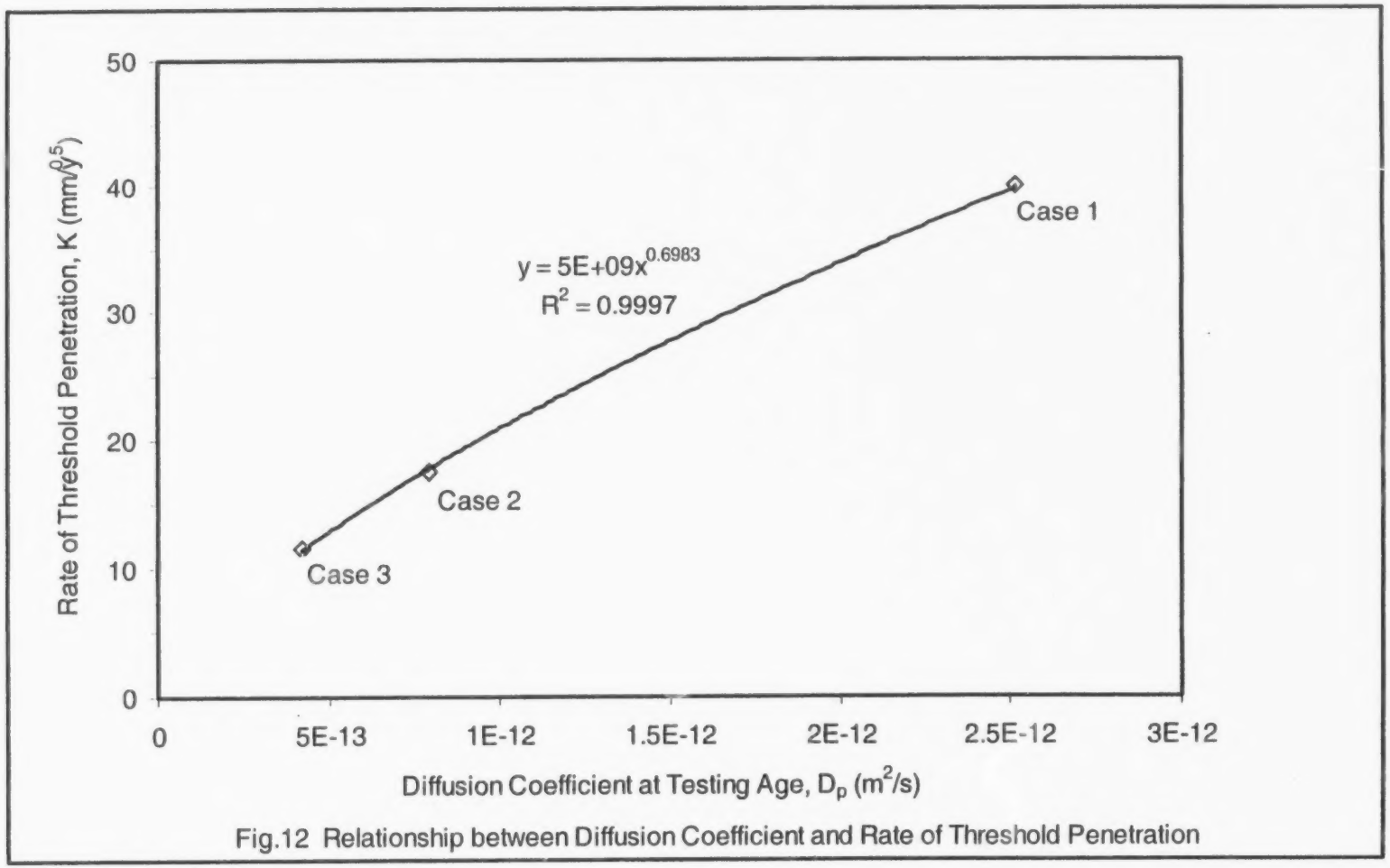


Fig.12 Relationship between Diffusion Coefficient and Rate of Threshold Penetration

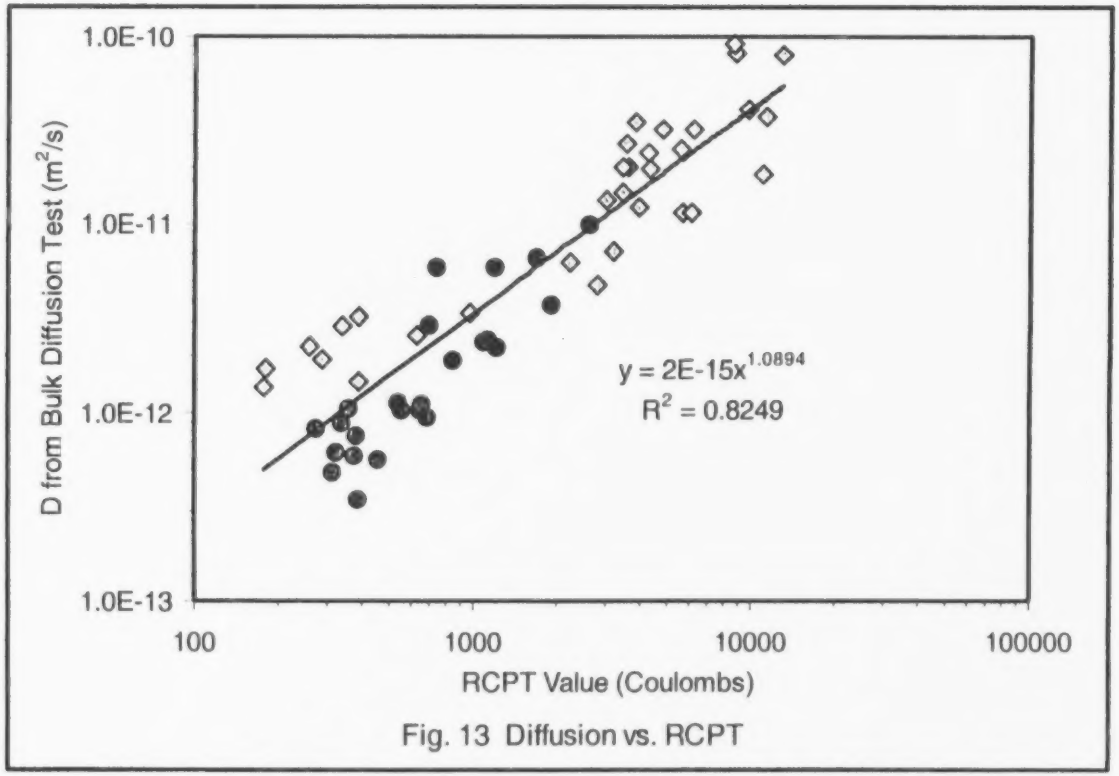


Fig. 13 Diffusion vs. RCPT

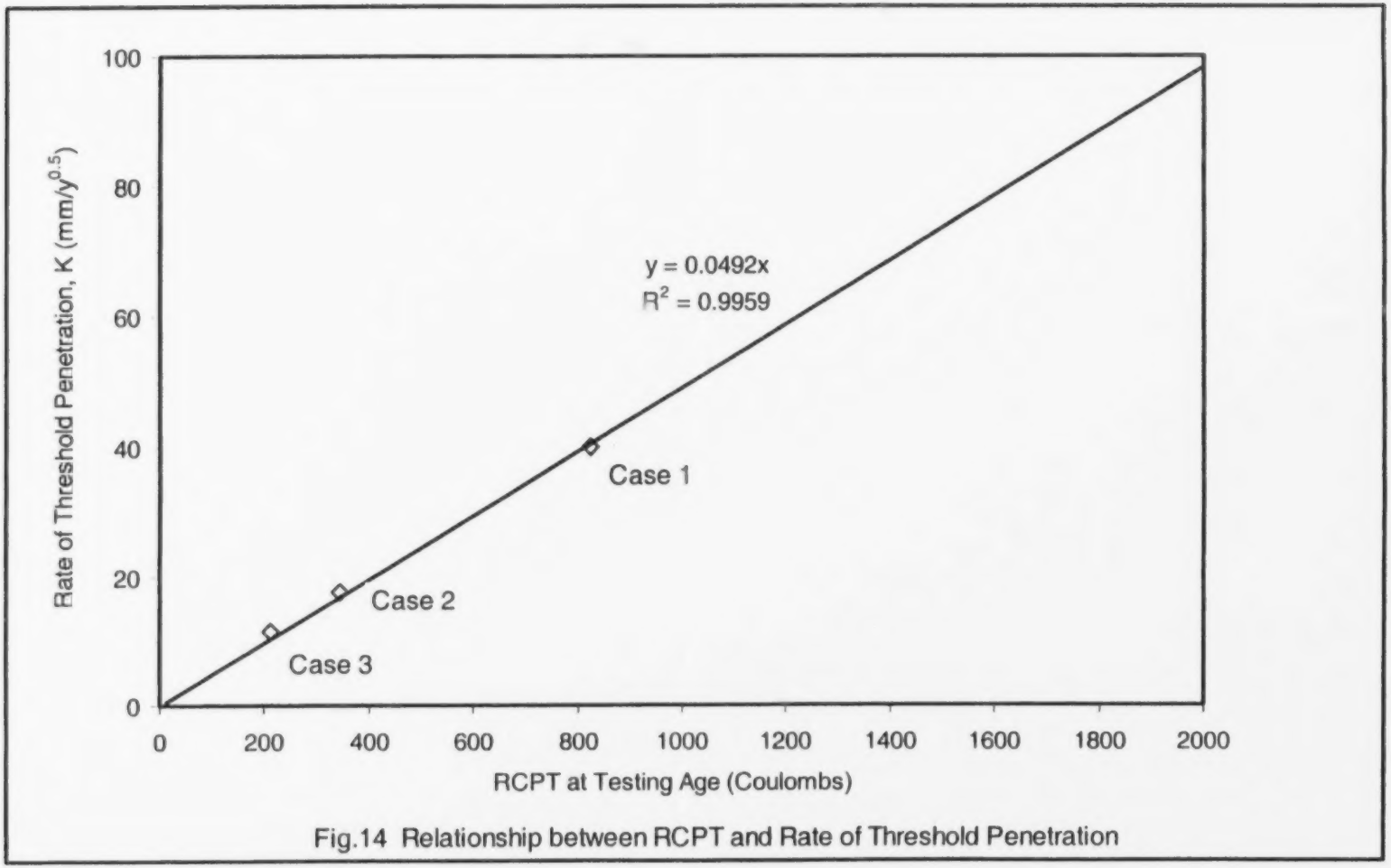


Fig.14 Relationship between RCPT and Rate of Threshold Penetration

APPENDIX C
Example Calculations for Service Life Predictions

APPENDIX C
Example Calculations for Service Life Predictions

Case 1: Protected Bridge Deck with Significant Chloride Content

Consider a 16-year-old bridge deck with a waterproofing membrane and asphalt overlay. The area of the bridge deck is 1600 m^2.

Table 1 of the Chapter 5 *Condition Survey Protocol* requires a total of sixteen 100-mm diameter cores to be taken only half of which have steel. This will result in a total of 16 chloride profiles and eight cover-depth measurements. Typical profiles for four of the cores are shown in Figure 1 below.

Time to Corrosion Initiation

From these profiles an estimate can be made of the depth of penetration, x_p, of the chloride threshold concentration (C_t = 0.025%). This can be done by direct interpolation of the data as shown above or, if the data show considerable scatter, by curve-fitting the error function solution of Fick's Second Law (Eqn. 2 in the Chapter 6 *Service Life Predictions*) to the data and then determining the appropriate x-value from the best-fit equation.

Figure 2 provides the values of x_p for all sixteen cores and shows the frequency distribution of these values. The distribution shown has a mean of $\overline{X} = 17.8$ mm and a standard deviation of $\sigma = 6.94$ mm. For the purpose of calculating the initiation period it is proposed that a value of x_p is selected such that the threshold concentration would only exceed the value 10% of the time. In other words $x_p = \overline{X} + 1.28\sigma$, which for the case shown in Figure 2 yields a value of x_p = 26.7 mm. This assumes a normal distribution of the measured values of x_p.

The measured depth of cover in the eight cores with steel produced the following data: 70, 55, 59, 63, 66, 58, 65, and 68. This distribution has a mean of $\overline{X} = 63.0$ mm and a standard deviation of $\sigma = 5.24$ mm. For the purpose of calculating the initiation period it is proposed that a value of x_d is selected such that the cover would only be lower than that value 10% of the time. In other words $x_p = \overline{X} - 1.28\sigma$, which for data given above yields a value of x_d = 56.3 mm. This assumes a normal distribution of the measured values of x_d.

The measured RCPT values for two 100-mm diameter cores sampled from the structure were 1150 and 1020 Coulombs giving an average of 1085 Coulombs. This yields a value of $k = 0.05RCPT = 54.25$

The time to initiation can be calculated as follows:

$$T_i = \left(\frac{x_d - x_p}{k} + \sqrt{T_a} \right)^2 = \left(\frac{56.3 - 26.7}{54.25} + \sqrt{16} \right)^2 = 21 years$$

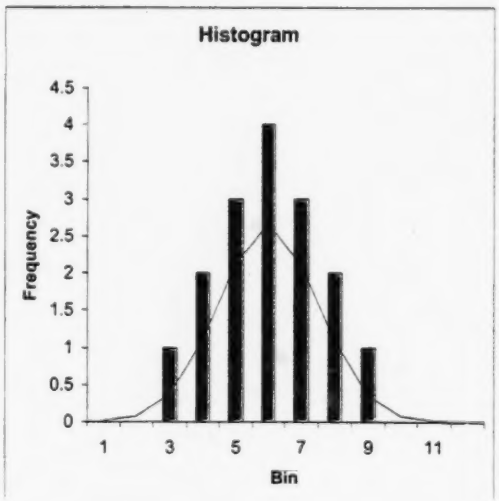
i.e., 5 years from the date of the condition survey.

Time to Propagation

Time to propagation is estimated using the measure of the number of defects and the results of the adhesion testing obtained from each ECR. The adhesion rating is required only for those ECR samples on which the density of coating defects does not exceed 40 per meter. Coating defects are comprised of the number of mashed areas (coating is damaged but steel is not exposed), number of bare areas (exposed steel areas), and coating holidays.

Per the condition protocol described in Chapter 5, a total of 8 cores containing two intersecting ECR were collected. This resulted in a total of 16 ECR evaluations. Typical data for 16 ECR are presented below.

Length of Bar, mm	# of Defects	Ni defects/m	T1	T2	Tp
96.9	14	144	0	8	8
96.9	15	155	0	7	7
94.4	16	170	0	5	5
94.4	15	159	0	6	6
96.9	16	165	0	6	6
96.9	15	155	0	7	7
96.9	18	186	0	4	4
94.4	15	159	0	6	6
96.9	16	165	0	6	6
96.9	15	155	0	7	7
94.4	12	127	0	9	9
94.4	13	138	0	8	8
96.9	17	175	0	5	5
96.9	18	186	0	4	4
96.9	19	196	0	3	3
94.4	17	180	0	5	5
				Average:	6
				Standard Deviation:	1.501824



The length of the bar and the number of defects for each are know, the number of defects per one meter length can be calculated by dividing the number of defects by the length of the bar in mm and multiplying by 1000. As the number of defects per meter for each bar is in excess of 40 per meter, T_1 is zero for each case in accordance with the following equation.

$T_1=2.5(40/N_i-1)$ for $N_i < 40$

$T_1=0$ for $N_i => 40$

$T_1=0$ for CR=> 4

T_2 for each bar is calculated using the following equation:

$T_2 = 16$ for $N_i < 40$

$T_2 = 19.25 - 13N_i/160$ for $40 \leq N_i < 200$

$T_2 = 3$ for $N_i \geq 200$

It is assumed that T_p can be represented by a normal distribution with a mean of 6 and a standard deviation of 1.5. Often data obtained from laboratory evaluation will appear to deviate from the

assumed normal distribution; this is a result of an insufficient number of data points. For the purpose of these calculations, it will suffice to use the mean and the standard deviation to describe the actual distribution of propagation time.

To pick a value for T_p, we select a value of T_p from the normal distribution such that values lower than it occur only 10% of the time. Statistically this value is represented as follows:

$T_p = \overline{T}_p - 1.28\sigma$ = 6-1.28(1.5)= 4.08 years ~ 4 years

Time to Damage

The estimated time to damage on the structure is calculated from Eqn. 1 in Chapter 6, Service Life Predictions, i.e. $T_i + T_p$ = 21+4= 25 years. i.e., 9 years from date of condition survey.

Action to be Taken

Carry out a Phase II Condition Survey in approximately 9 years time.

Case 2: Protected Bridge Deck with Isolated Chloride Content

Consider a 20-year-old bridge deck with a waterproofing membrane and asphalt overlay. The area of the bridge deck is 5000 m^2.

Table 1 of the Chapter 5 *Condition Survey Protocol* requires a total of thirty 100-mm diameter cores to be taken only half of which have steel. This will result in a total of 30 chloride profiles and fifteen cover-depth measurements. In this case, no significant chlorides are found in 24 of the cores (i.e. $C_t < 0.025\%$ at all locations) and profiles for the remaining 6 cores are shown in Figure 3 below.

These data represent a distribution with a mean of $\overline{X} = 2.5$ mm and a standard deviation of $\sigma = 5.73$ mm. For the purpose of calculating the initiation period it is proposed that a value of x_p is selected such that the threshold concentration would only exceed the value 10% of the time. In other words $x_p = \overline{X} + 1.28\sigma$, which yields a value of x_p = 9.83 mm. This assumes a normal distribution of the measured values of x_p (which will not actually be the case when there is isolated breakthrough of chlorides).

Since $x_p < 10$ mm, the recommendation is that the no further testing be carried out and the that the structure be re-evaluated after a further 15 years.

NOTE: In this report the chloride content of the concrete (by mass) is the acid soluble chloride content less the background chloride content of the structure components.

X (mm)	Core 1	Core 2	Core 3	Core 4
5	0.054	0.125	0.241	0.101
10	0.061	0.153	0.132	0.099
15	0.02	0.122	0.08	0.075
20	0.015	0.032	0.034	0.031
25	0.01	0.02	0.022	0.029
30	0.009	0.011	0.004	0.024
35	0.012	0.017	0.008	0.029
40	0.013	0.009	0.014	0.024
45	0.016	0.008	0.008	0.008
50	0.005	0.012	0.008	0.01
X_p	14	23	24	40

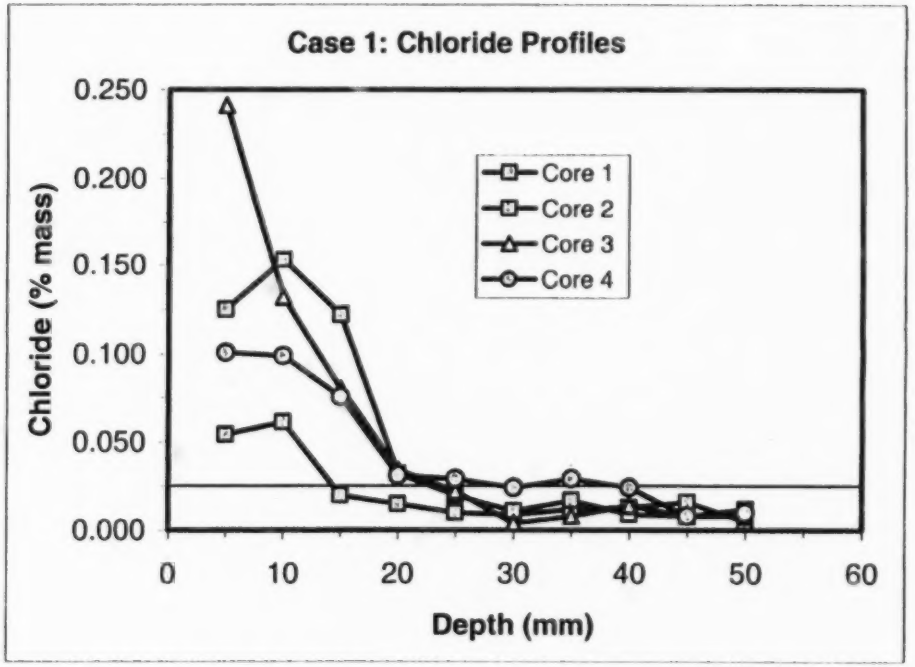


Figure 1

	X_p (mm)
Core 1	14
Core 2	23
Core 3	24
Core 4	39
Core 5	16
Core 6	12
Core 7	18
Core 8	15
Core 9	17
Core 10	21
Core 11	9
Core 12	15
Core 13	19
Core 14	16
Core 15	12
Core 16	14
Mean	**17.8**
SD	**6.94**

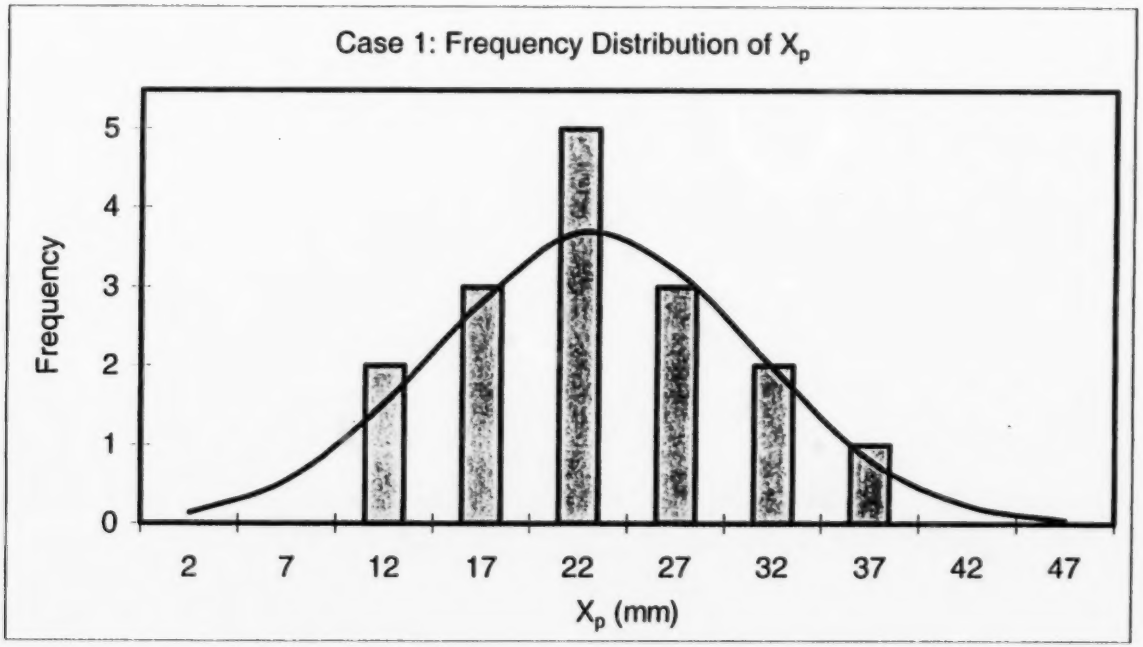


Figure 2

X (mm)	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
5	0.081	0.115	0.020	0.061	0.040	0.027
10	0.061	0.071	0.035	0.025	0.015	0.008
15	0.020	0.086	0.031	0.010	0.003	0.007
20	0.015	0.030	0.004	0.006	0.005	0.003
25	0.010	0.016	0.003	0.004	0.008	0.004
30	0.009	0.003	0.006	0.007	0.004	0.005
35	0.006	0.002	0.008	0.003	0.002	0.003
40	0.013	0.001	0.008	0.000	0.004	0.001
45	0.003	0.008	0.002	0.008	0.001	0.001
50	0.005	0.009	0.000	0.004	0.001	0.002
X_p	14.5	22	16	10	8	5.5

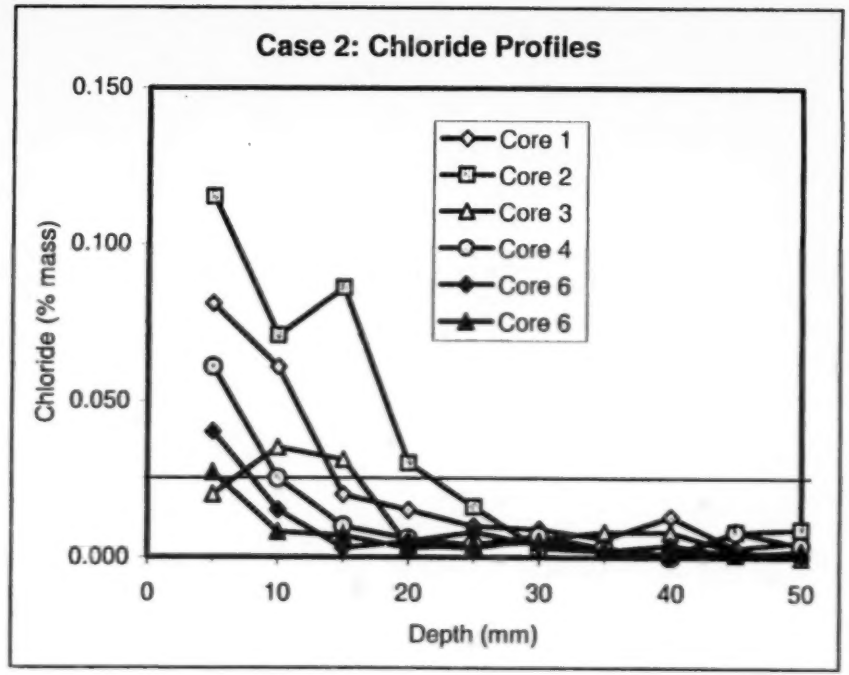


Figure 3

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